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No. 7. 1935-36.
JUNE 1936.

ORDINARY MEETING.

24 March, 1936.

Mr. JOHN DUNCAN WATSON, President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

NORMAN GARSTANG ELLIOT, M.Sc. (*Leeds*).
HAROLD HOBSON, B.Sc. (Eng.) (*Lond.*).
ABDUL AZIM ISMAIL.
JOHN WEYMAN MASON, B.Sc. (Eng.) (*Lond.*).
JAMES LAWRENCE SANDERS, B.Sc. (Eng.) (*Lond.*).
RALPH ALEXANDER WHITSON.

And had admitted as

Students.

ERIC SYDNEY ALLWRIGHT, B.Sc. (Eng.) (*Lond.*).
BASIL HOFMEYR BRADFIELD, B.Sc. (*Cape Town*).
WILLIAM ROBERT CLEMENTSON, B.Eng. (*Liverpool*).
HENRY CRISWELL, Jun., B.Sc. (Eng.) (*Lond.*).
PESTONJI JEHANGIRJI DARUKHANAVALA, B.E. (*Bombay*).
ERIC EDWARD DEAN, B.Sc. (*Birmingham*).
SHEPARD DIXON, Jun.
HUBERT MORNINGTON DOCWRA.
JOHN HERBERT EDWARDS, B.Sc. (*Cape Town*).
TREVOR VINCENT MARTIN, B.Sc. (Eng.) (*Lond.*).
GEORGE LESLIE MOORE.
DAVID MORRIS.
DOUGLAS DALGARNO MURRAY, B.Sc. (*Aberdeen*).
GEORGE BOSWELL NAPIER.
ERNEST FREDERICK NICOLAY, B.Sc. (Eng.) (*Lond.*).
NATHAN RAYMAN, B.Sc. (*Birmingham*).
GEORGE GORDON ROSE.
HENRY KENNETH ROSEVEARE, B.A. (*Cantab.*).
NORMAN TAYLOR, B.Sc. Tech. (*Manchester*).
WILFRED ARSCOTT TILBROOK, B.Eng. (*Sheffield*).
KENNETH ALLEN WALKER.
LESLIE WHITELEY.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5036.

"The Superstructure of the Island of Orleans Suspension Bridge, Quebec, Canada."

By SHEWELL REGINALD BANKS, M. Eng., Assoc. M. Inst. C.E.

*** The Paper and Discussion, together with the Correspondence, will be published in the October Journal.—SEC. INST. C.E.

ORDINARY MEETING.

7 April, 1936.

Mr. JOHN DUNCAN WATSON, President, in the Chair.

The Scrutineers reported that the following had been duly elected as

Members.

ALFRED HILL.
AUBREY DUNCAN MACKENZIE.
FUKYIUN KENNETH SAH.

Associate Members.

OTHMAN FRANK BLAKEY, M.E. (*Queensland*), M.E. (*W. Australia*).
EDWARD NEVILLE CLARKSON, B.Sc. (*Durham*) (Stud. Inst. C.E.).
ROBERT SEPTIMUS FRIAR EDWARDS (Stud. Inst. C.E.).
HAROLD DONOVAN GAUNTLETT (Stud. Inst. C.E.).
FREDERICK LESLIE GREEN, B.Sc. (Eng.) (*Lond.*) (Stud. Inst. C.E.).
RICHARD ERNEST HETHERINGTON (Stud. Inst. C.E.).
JOSEPH JACKSON, B.Sc. Tech. (*Manchester*) (Stud. Inst. C.E.).
IVOR ROBERTS MORGAN, B.Sc. (*Wales*) (Stud. Inst. C.E.).
HAROLD MOYS, B.Sc. (Eng.) (*Lond.*) (Stud. Inst. C.E.).
WALTER MORRIS SCOTT, B.Eng. (*Sheffield*) (Stud. Inst. C.E.).
CECIL TINKLER, B.Sc. (*Durham*) (Stud. Inst. C.E.).
WILLIAM EWYAS WALLEY, B.Sc. (*Wales*) (Stud. Inst. C.E.).

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

Paper No. 4989.

"Corrosion of Iron and Steel."

By Sir ROBERT ABBOTT HADFIELD, Bart., D.Sc., D.Met., F.R.S., M. Inst. C.E., and SIDNEY ARTHUR MAIN, B.Sc.

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Introduction.

Since one of the Authors presented his Paper on the " Corrosion of Ferrous Metals " to The Institution in 1922,[1] the attention given to the problem of corrosion, which was already considerable both among representative bodies and individual investigators, has rapidly increased. It is therefore one of the objects of the present Paper to review the activities since this date, and to give some account of the further knowledge gained of the effects and nature of corrosion.

It is proposed to devote particular attention to the work of the Committee on the Deterioration of Structures exposed to Sea-Action, which has now been at work for nearly 20 years, and the principal object of the present Paper is to describe the twenty-two important specimens which were furnished, at the wish of the Committee, after the research was originally planned.

A later section of this Paper is devoted to the results of various exposure trials with which the Authors have been concerned, and they feel that these will be of interest to those members of The Institution who are concerned with harbour-works or other marine structures. These experiments dealt mainly with the behaviour of various steels subjected to marine corrosion at Ipswich, the Thames Estuary, the North Sea, and the Gulf of Paria in Trinidad.

Modern Researches on Corrosion.

The Authors believe that it will be of interest to call attention to the following statements as showing the importance of iron and steel, and the necessity for the study of corrosion.

Dr. F. N. Speller, Director of Research at the National Tube Company, Pittsburg, has made a careful estimate of the total amount of steel in use in the world, and finds it is about 1,200 million tons, 700 million tons of which are probably in use in the United States; of that amount, he further estimates that about 75 per cent. requires protection in order to be used.

Another American authority, Dr. Colin G. Fink, Head of the Division of Electrochemistry, Columbia University, New York City, states that, without doubt, the basic metal in the world, from an engineering point of view, is steel, and its application will undoubtedly continue to grow, and greatly exceed the already large figure of 119 million tons for the year 1929. He also draws attention to the statements that copper and aluminium will replace steel, and he points out that steel has certain properties which render it superior to these metals.

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[1] Sir Robert Hadfield. Minutes of Proceedings Inst. C.E., vol. ccxiv, p. 83.

While the study of the corrosion of iron and steel may still show imperfect, and sometimes not altogether definite, results, each new fact gained represents an advance in knowledge. The services of the metallurgist and the chemist in this respect cannot be rated too highly. Perhaps the most striking feature of the investigation into the question of corrosion has been the way in which, since 1922, the work has been placed in the hands of special committees, who have had to correlate information from many sources. In the past, on many occasions the effects of corrosion on iron and steel have been recorded in much detail, but in many cases there has been no proper record of the precise conditions under which such corrosion has taken place, or of the true nature and origin of the material concerned. The collaboration, in the form of committees, of metallurgists, engineers and expert observers renders it possible to obtain information upon which sound conclusions can be based. The initiation of adequate researches on corrosion in any particular branch often requires a very comprehensive scheme and large resources, which can only be brought into effect by the combined efforts of several individuals. A list of the Committees known to the Authors which are active at the present time in dealing with the subject of corrosion is given in Appendix I.

International Co-operation.—The problem of corrosion is one which concerns all countries, and the campaign against its ravages calls for the fullest possible international co-operation. The free intercourse which exists between scientific investigators in different countries does much to secure such co-operation, and the publications of the various scientific societies are often enriched by communications from foreign sources. Special mention should be made in this connection of a paper by Dr. J. Montgomerie and Mr. W. E. Lewis, on "Corrosion in Hulls of Merchant Vessels." [1] Many prominent authorities, both British and foreign, and representing both practical experience and laboratory research, took part in the discussion, which constitutes an excellent symposium. Incidentally that paper, while it is concerned with ships, is nevertheless of interest to engineers dealing with marine structures, from the insight which it gives into the corrosive action of sea water as ascertained from experience obtained in the course of Lloyds' surveys.

Still more direct co-operation was achieved in the "International Convocation on Corrosion, and a Faraday Celebration" which was held at Chicago in September, 1933, at which a large number of papers were presented from investigators in many countries.

Utility of Practical Exposure Trials.—There can be no doubt that

[1] Trans. Inst. Engineers and Shipbuilders in Scotland, vol. lxxv (1931–1932), p. 391.

great progress has been made as the result both of collective efforts and of individual investigation. From practical exposure tests, many results of an authentic nature have been accumulated, which engineers may safely use for their guidance. For the present, and probably for some time to come, there is nothing in this respect to take the place of such tests, although it may be urged that the results obtained are peculiar to the particular set of conditions operating, and that the amount and character of the corrosion observed is the result of a combination of factors, each of which may be different in another location or under other circumstances.

Laboratory tests intended to simulate practical conditions of corrosion have undergone considerable improvement during the period under review, particularly in careful attention to details, and it has become possible to say what would be the behaviour of any particular metal, both as regards the amount of wastage and the character of the corrosive attack, provided that the composition of the metal and the physical conditions are specified. The conditions in such laboratory tests can, however, only be of a comparatively simple character, and are not therefore usually related in any definite way to practical conditions, which are almost invariably more complicated. It is difficult, for example, to simulate in the laboratory the organisms which abound in the sea, and the effects of molluscs or other fauna which adhere to steel structures. Laboratory tests of this kind cannot, therefore, effectively take the place of actual exposure tests under service conditions, although it must be said in their favour that much information of value as to the nature of the processes of corrosion has been derived from laboratory tests.

As further knowledge of the effects of corrosion has been gained it has become clear that, owing to the complexities involved in almost every practical case, very little could be definitely inferred as to the true nature of corrosive action itself by merely watching the progress of the corrosion as it ordinarily occurs. It has been possible from such observations to formulate theories, some of which have survived. For example, in many cases there is some certainty of the acceleration of corrosion by electrolytic action, due to the contact of dissimilar metals, and an electrolytic theory of corrosion came into being. While this theory does not, as some of its exponents at one time thought, explain the whole cause of corrosion, electrolytic action is now regarded as an essential stage in some of the more modern theories such as the "anodic" or differential-aeration theory. It is important to realize, however, that the earlier theories were not based alone on observations of the progress of corrosion as it occurs in nature, but also on correlated scientific knowledge derived from laboratory experiments. A growing

appreciation of the importance of such laboratory experiments, and the position which they occupy in the campaign against corrosion, has been a feature of the more recent work on the subject.

Dr. G. D. Bengough, in his Jubilee Memorial Lecture (1932–33) to the Society of Chemical Industry,[1] dealt with some of the many factors which influence corrosion. Those factors which operate in the case of salt solutions were comprised in the following list :—

Metallic Factors.	Environmental Factors.
(1) The electrode potential, which is influenced by :	(1) The temperature.
(a) The state of aggregation.	(2) The partial pressure of oxygen.
(b) The state of strain.	(3) The supply of oxygen.
(c) The crystal orientation.	(4) The concentration of hydrogen ions.
(2) The hydrogen over-potential.	(5) The concentration of metal ions.
(3) Metals in solid solutions.	(6) The concentration and specific nature of other ions present.
(4) The presence of different phases.	(7) The nature and distribution of corrosion-products.
(5) The chemical reactivity.	(8) The electrical conductivity of the solution.
(6) The surface-condition.	

From these figures it will be seen that, even when the corroding medium is specified and the metal is pure, the corrosion experienced is largely determined by the physical state of the metal, and by the surrounding conditions.

It is further clear from these figures that the theory that each metal possesses an inherent property, termed "corrodibility," which can be ascertained and tabulated with its other properties, such as its specific heat and its conductivity, is entirely erroneous.

Most processes of corrosion occupy several stages; for example, with iron in a solution of common salt, the succession of stages is as follows :—

(1) A difference of electrical potential between two areas on the surface sets up an electrolytic current passing through the liquid between those areas, and returning through the metal.

(2) At the area constituting the positive pole or anode, chlorine ions in solution, being discharged, attack the metal, forming ferrous chloride which is dissolved.

(3) At the same time caustic soda is formed near the negative (cathodic) area, which is similarly soluble. Hence at this stage there is still no visible solid corrosion-product.

(4) The ferrous chloride and caustic soda react with each other where they meet in solution, and produce ferrous hydroxide.

[1] "Corrosion of Metals in Salt Solutions and Seawater," Journal Soc. Chem. Ind., vol. 52 (1933), pp. 195, 228, 275.

(5) By the action of the dissolved oxygen, the ferrous hydroxide is changed to brown hydrated ferric oxide, which is the insoluble product known as rust.

From the above list of stages, it will be seen that the study of the practical aspects of corrosion is one in which the engineer must co-operate to the full with the chemist.

Dr. Bengough's researches have established a technique which seems to be capable of unlimited use when applied to the investigation of the many variations possible, whether of metal, corroding medium or conditions of environment.

Special mention should be made of protective oxide films and the part which they play when dealing with the discoveries made in corrosion research in recent years. Under exposure to air, it has been definitely established that the common metals acquire a pellicle of oxide by the direct chemical attack of oxygen. This film is extremely thin, not more than 1·6 millionths of an inch in thickness, and is therefore quite transparent, and it is not surprising that its existence, although suspected, was not definitely established until a few years ago, when Dr. Ulick Evans and his colleagues succeeded in separating it from the metal. The significance of these films is that they actually provide an armour or first line of defence for the metal against corrosion, and this armour has first to be penetrated before the ordinary electrochemical processes of corrosion can get to work. With certain metals and under suitable conditions, as has been clearly shown by Dr. W. H. J. Vernon, the protection thus afforded can be highly effective, and the film has also the power, to a limited degree, of repairing itself when it is damaged. The "stainless" chromium steels are believed to owe their specially-resistant properties to the impermeable character of the film which they acquire in this way. Unfortunately, in the case of iron and ordinary steel the film is permeable to the corrosive agencies found in ordinary water and sea-water, and it does not therefore offer an effective obstacle against ordinary forms of corrosion.

Opinions differ as to the precise nature of the roles played by oxygen in corrosion processes. It is generally agreed that oxygen directs chemical action on to the metal, forming a film, and further that it enters chemically into the series of reactions which result in the formation of rust. Opinions differ, too, as to the extent to which electrolytic action by oxygen, through what is known as "differential aeration," is effective in explaining the peculiarities in corrosion; electrolytic action certainly plays a part in the corrosion process, but the circumstances which may set up such action are many and varied. Among the simplest are the direct contact of dissimilar metals, or a non-homogeneous structure in the metal;

this latter may only be non-homogeneity of its physical condition. A further and less apparent circumstance which may promote an electrolytic current is non-homogeneity of the corrosive solution. Dr. Ulick R. Evans in his James Forrest Lecture [1] explained certain effects observed both in his experiments and in practical cases in this way by the differences in the degree of aeration of the liquid, brought about either by the surrounding conditions, or due to the local consumption of oxygen in the corrosion process, or to both of these causes.

The reality of the differences of potential which can exist between different points on the surface of a metal and so give rise to electrolytic currents was well expressed by Dr. Colin G. Fink in the discussion of a paper by Dr. W. H. J. Vernon on "The Role of the Corrosion Product in the Atmospheric Corrosion of Iron," at the International Convocation on Corrosion, held at Chicago in 1933. Dr. Fink stated that the theoretical electrolytic couples referred to by Dr. Vernon actually existed; Dr. Kenny, in the laboratory of Columbia University, had measured these point-to-point couples, and it was of great interest to see that they actually existed, and that the explanation of the process of corrosion was not dependent on a hypothesis. The pressure of a pin on the surface of a piece of steel will make the area under the pin-point into an anode, and a particle of dust will make it anodic as compared with its surroundings.

Opinions differ as to the importance of the part played by oxygen in electrolytic action, particularly as regards pitting, for which "differential aeration" seems to afford a ready explanation. According to this view, pits are liable to be formed where an obstruction, which may be a deposited particle, prevents free access of oxygen to the surface, which becomes, therefore, locally anodic to the rest of the surface. When the pit is actually formed, it tends to develop because the corrosion products contained in it again hinder the access of oxygen, or, alternatively, the oxygen used up by corrosion is not so readily replenished under the stagnant conditions which exist in the pit, as it is elsewhere. Dr. Bengough, however, while recognizing that "differential aeration" may be operative in some circumstances, inclines more to what he terms a "film-distribution view" of the corrosion process, by which the distribution of corrosion is believed to be controlled by several factors, and in which the direct protective action of films of corrosion-products plays a prominent part.

The discovery by Dr. Vernon that deposited dust-particles can play a prominent part in promoting corrosion, causing, in the presence of sufficient moisture, definite corrosion at the points

[1] Minutes of Proceedings, Inst. C.E., vol. 234 (1932), p. 445.

where they are located, is a further outstanding feature of modern research in the subject of corrosion. Dr. Vernon attributes the pitting that occurs under these dust-particles mainly to the direct action of the chemical substances, usually sulphates of ammonia, which they contain.

A further important discovery of recent times, and one which established a direct and previously-unsuspected effect of corrosive action on the strength of metals, concerns what is known as "corrosion fatigue." The weakening of structures due to actual loss of metal through corrosion is obvious, but it was not so clear that corrosion to a degree short of that which causes a reduction in the section of metal can cause a marked reduction in strength, in many cases by as much as 50 per cent. The strength-factor most influenced in this way, as shown by the researches of Mr. D. J. McAdam, Junr., and Dr. H. J. Gough, is resistance to those repeated stresses which, if excessive, cause failure by fatigue. The explanation appears to be that such stresses have a disruptive effect on the protective oxide film, allowing the corrosive medium to have access to the metal at various points. Minute pits or channels are formed in the surface of the metal, and these operate in the very sensitive way in which surface blemishes depreciate the fatigue-resisting properties of metals.

Dr. Ulick Evans, in his James Forrest Lecture,[1] pointed out that it was probable that the number of failures which could be partly attributed to chemical influences was larger than was commonly suspected, having regard to the insidious manner in which corrosion often worked. It might even cause disruption of a structure by the formation of rust in the joints due to the greater volume of the rust as compared with the metal from which it is formed, and he indicated the dangers which would arise with any tendency to reduce factors of safety. He said: "The recent triumphs of mathematics may occasionally tempt engineers to reduce the dimensions of structural components, but, in some cases where this would be justifiable if mechanical factors alone are considered, the reduction may nevertheless cause an increase of peril from chemical forces, which are, in general, more apt to attack what is slender or flimsy rather than what is stout and rigid. Any liability to bending or oscillation, however harmless mechanically, increases the probability of the inception of attack, by cracking either the paint-coats, the mill-scale or the invisible skin which lies next to the metal."

The facts and views here mentioned are only some of the more important of those which are the outcome of research on corrosion in recent years, but, even in the present brief outline, mention should

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[1] *Ibid.*, p. 448.

also be made of the attention which is being given in a large number of investigations to the subject of protective paints and coatings, as the result of which the principles underlying their correct choice and application are now becoming much more fully understood.

The Research Work of the Sea-Action Committee of The Institution.

The Paper referred to [1] gave some account of the formation in 1916 of the Committee on the Deterioration of Structures exposed to Sea-Action, and particularly as regards the planning of the extensive research which they had undertaken on the marine corrosion of ferrous metals. The Committee owed its origin largely to the late Sir John Wolfe Barry, K.C.B., F.R.S., M. Inst. C.E., who, in the past, has probably done more to help civil engineering than anyone else.

Sir John worked particularly in the direction of improving harbour- and dock-construction, and a little note, still in the possession of one of the Authors, was passed to him one day by Sir John at a Council meeting of The Institution held early in 1915. On this note he had written the following words: "Did you find out anything about non-corroding or non-staining steel, and its composition?" This was probably, if not the beginning, at any rate one of the earliest episodes which resulted in The Institution taking such an active interest in the important subject of corrosion. Correspondence resulted, in which Sir John was informed of the history and characteristics of the 12–14 per cent. chromium steel, in which he showed much interest, and in which he asked whether this non-corrodible steel could be used for dock-gates and other similar purposes.

In view of the initiation of the Committee as long back as 1916, it may be of interest to add the following short statement with regard to the most important dates in connection with this research. The Committee's first meeting took place on 29 June, 1916, the Chairman being the late Sir William Matthews, K.C.M.G., Past-President Inst. C.E. Fourteen Interim Reports have been issued, one each year, commencing with 1920. A full report, summing up the whole of the work in all the four sections, and including some brief introductory remarks prepared by Sir Robert Hadfield regarding the section dealing with iron and steel, has also been published.

Mention should be made of the laborious work and help given by the Secretaries of the Committee, namely, the late Mr. P. M. Crosthwaite, B.A.I., M. Inst. C.E., from 1916 to 1926; Professor J. Purser, M.Sc., B.A.I., from 1926 to 1933; and the present Secretary, Professor S. M. Dixon, M.A., B.A.I., M. Inst. C.E., who

[1] *Ante*, p. 4.

has also in the past furnished reports which have been of great value to the work of the Committee. During the course of this long research, already spread over nearly 20 years, much help has been afforded to the Committee in its earliest work by the former Secretary of The Institution, Dr. J. H. T. Tudsbery, D.Sc., M. Inst. C.E., now Honorary Secretary, and later by the present Secretary, Dr. H. H. Jeffcott, B.A., B.A.I., Sc.D., M. Inst. C.E., both of whom have on many occasions most kindly assisted the Authors, who desire to take this opportunity of returning their cordial thanks to each of them.

The present Paper, as in the case of that referred to on p. 4, deals specifically with that section of the investigations devoted to the examination of the corrosion-resisting qualities of the various iron and steel specimens submitted to the many tests prescribed and carried out by the Committee.

The other important portions of the researches are dealt with by :—

(1) The Sub-Committee devoted to the study of preservative coatings for steel ;
(2) The Sub-Committee devoted to the study of concrete. In this research Dr. R. E. Stradling, M. Inst. C.E., and the staff of the Building Research Station at Watford, and Mr. C. Percy Taylor, M. Inst. C.E., Chief Engineer to the Associated Portland Cement Company, Ltd., have given much practical assistance ;
(3) The Sub-Committee on creosote tanks ;
(4) The Sub-Committee for the final report.

The Chairman of each of these four Sub-Committees is Mr. M. F-G. Wilson, M. Inst. C.E., and the full information regarding the respective activities of these Sub-Committees and the important work they have carried out is fully set out in the fourteen official Interim Reports issued from 1920 to 1934.

At the commencement of this research, Sir Robert Hadfield was asked by Sir John Wolfe Barry and other members of the Committee to recommend an expert in the study of corrosion, and he suggested the name of Dr. J. Newton Friend, who is well known for his able research work on the subject of corrosion. His help has been found to be most valuable and the Authors, who have also consulted him from time to time, take this opportunity of thanking him most cordially for various suggestions and information.

Dr. Friend personally supervised, on behalf of the Committee, the final preparation of the nine hundred and eighty specimens and their packing for dispatch to the various stations. This required frequent visits to the research laboratories of the Hecla and East

Hecla works of Messrs. Hadfields, Ltd., Sheffield, where this work was carried out, as it included the laborious marking and weighing of each individual specimen, and in some cases their assembly by riveting and bolting into the various required forms and combinations, and also the mechanical and other physical tests. The whole of these tests, and also the numerous chemical analyses, micro-examinations, and much other work required from time to time by the Committee, have been carried out by the Authors entirely in the research laboratories at Sheffield.

Dr. Friend and his staff have also done all the work of preparing, examining, and weighing the corroded specimens at the conclusion of the period of exposure, which was either 5 or 10 years. The work of recording the weights in a research of this nature, extending over a lengthy period of time, is of particular importance, as if errors were to occur, the results of the research would be worthless. The excellent concordance which has been found in the results obtained by the Committee is undoubtedly to be attributed in a large measure to the careful and painstaking manner in which these duties have been discharged.

Work Performed.—The original plan for the research on ferrous materials, for the details of which reference should be made to the Paper cited on p. 4, comprised fourteen types of material, with the addition of a further material, as described later, representing chromium steel of low carbon-content. The full list is as follows :—

Section I.—Irons, rolled and forged :—wrought iron ; Swedish charcoal iron ; ingot iron.

Section II.—Carbon steels :—mild steel, with low manganese and high sulphur and phosphorus contents ; mild steel, with 0·7 per cent. manganese ; medium carbon steel with low sulphur and phosphorus contents ; 0·40-per-cent. carbon steel.

Section III.—Special steels :—mild steel with ½ per cent. copper ; mild steel with 2 per cent. copper ; 3½-per-cent. nickel steel ; 36-per-cent. nickel steel ; 13-per-cent. chromium—hard grade ; 13-per-cent. chromium—soft grade.

Section IV.—Cast irons :—cold-blast cast iron ; hot-blast cast iron.

The carbon and special steels were produced for the Committee by Messrs. Hadfields, Ltd., of Sheffield, altogether one thousand three hundred and fifty specimens being prepared, of which nine hundred and fifty-five were allocated in the original scheme to the various exposure and laboratory tests. The exposure of four further bars of chromium steel, and of eighteen specimens of an additional material representing chromium steel of lower carbon content, with the provision of the necessary bars for mechanical and physical tests, has since increased the total to nine hundred and eighty.

Particulars of the manufacture of the various materials, with details of the numbers of bars prepared from them and their allocation for the purposes of the research, are given in Appendix II, Table II. From this it will be seen that, of the number originally prepared, some four hundred specimens still remain for any further use.

The specimens, in triplicate as regards the main series of tests, were distributed to stations at Auckland, Colombo, Halifax (Nova Scotia), and Plymouth, where, under the instructions of members of The Institution who were harbour-engineers there, they were assembled in suitable concrete frames and submitted to the prescribed conditions of exposure for periods of 5, 10 or 15 years.

The comprehensive programme of investigation comprised many different tests, including the effects of total and intermittent immersion in sea-water, exposure to marine atmosphere, and corrosion in fresh water as well as the variations of corrosion arising from geographical situation. Other tests included the influence of original scale on the surface, the influence of strain on the metal, and the contact of dissimilar metals.

A full series of mechanical tests was carried out on the specimens in both the untreated condition as exposed, and also after heat-treatment, which required, with the additional specimens, two hundred and thirty-eight test-pieces. The tests also included twenty-six complete analytical tests and fifty-eight micro-examinations, as well as eighty-seven Brinell hardness-tests.

The completion of the Committee's programme will not be reached until the period allocated for the longest term of exposure, namely, 15 years, is exhausted. Meanwhile, however, much valuable information has been obtained from the examination of the specimens exposed for 5- and 10-year periods respectively, and this will be found in the various Interim Reports of the Committee.

A separate analysis, by the Authors, of the Committee's results, so far as they concern the relative merits of the different ferrous materials, and their response to the widely-varying conditions to which they have been subjected, is presented later in this Paper. This is not intended to replace the Committee's own published reports, but to supplement them, in an endeavour to assign actual comparative figures and to investigate the importance of the different factors promoting corrosion and pitting, according to climate and method of exposure.

The specimens which have been added to the main series undergoing investigation by the Committee, the reasons for their inclusion, and the particulars of their preparation, will now be described. These additional specimens comprise two separate batches, representing twenty-two specimens in all.

Polished Specimens of Chromium Steel.—Early observations of the behaviour of the 13-per-cent. chromium steel specimens in the routine visual examination at Auckland gave the impression that this material was corroding more rapidly than the ordinary irons and steels. While it was realized that the appearance of the specimens observed in this way might be misleading, and that the only true indication of their merits would be given by eventual examination with the products of corrosion removed, it was thought that the presence of scale might be exerting a particularly marked influence on the relative behaviour of the material. This steel, in order to display its full non-corrodible qualities, must usually, in addition to being heat-treated, have a surface as smooth and as clean as possible. It was thought desirable, therefore, to include in the tests, for comparison, bars of this same chromium steel from which the scale had been removed.

For this purpose four bars, numbered J87 to J90 inclusive, which had already been softened by heating to 600° C. and cooling in the furnace, were ground all over until they were entirely free from scale, and were then well polished. Holes were drilled at their ends in accordance with the scheme, with the classification-letter "J" and individual number of the bar, for identification purposes. As the object was to ascertain the effect of the scale, the bars were not specially heat-treated. Except for the difference in the nature of the surface their physical condition is thus the same as that of the bars of the same steel in Section III of the main research, and the mechanical test data and particulars of the microstructure shown for the latter are equally representative of these additional specimens.

On completion of their preparation in December, 1922, the bars J87 to J90 were allocated to Plymouth for exposure under the four different conditions planned in the main research, namely, aerial exposure, half-tide exposure, and total immersion in both sea-water and fresh water; by comparison with the similar but unpolished bars included in the original research, at intervals and at the end of a prescribed term of exposure, the influence of scale on the corrosion of this steel could thus be ascertained.

Chromium Steel of Softer Grade.—Subsequently to the planning of the research, there came into existence a modified type of chromium steel of a softer and more ductile kind. This material is generally known as "stainless iron" which, however, is rather a misnomer, as it is essentially a steel, and only differs from "stainless steel" in the amount of its carbon content. Whereas the "steel" may contain carbon to the amount of 0·20 to 0·40, or even in some cases as high as 0·60 per cent. in the "iron" the percentage of carbon is kept as low as possible, usually not amounting to more

than 0·15 per cent. For convenience in this research the two types are distinguished as hard and soft grades of the chromium steel, containing from 12 to 14 per cent. of chromium.

As the lower tenacity and greater ductility of the soft grade render it, from the mechanical point of view, more suited to the structures concerned in harbour-works, the Committee considered it desirable to include specimens of this material in the research, and a limited series of specimens was therefore provided, as shown in Appendix II, Table I.

Preparation of Specimens.—The steel was prepared by Messrs. Hadfields, Ltd., in an electric-arc furnace, and cast into 6-inch ingots, one of which was rolled into bars 3 inches by ⅜ inch which were cut into 24-inch lengths, so as to provide bars comparable with those of the main research. The specimens, in the form of plain bars, were exposed at Plymouth singly, under aerial, half-tide and total immersion conditions, and in direct comparison with the specimens in the main research.

Opportunity was taken to investigate further the effect of the condition of the surface on the corrosion-resisting qualities of steel of this type. The specimens therefore included bars with three different conditions of surface, as follows :—

(*a*) With the rolling- or treatment-scale left on, similar to the majority of the specimens in the original research.

(*b*) With the scale removed by pickling in dilute *aqua regia*, the surface being otherwise untouched.

(*c*) With polished surface, the bars being finally brought to a high degree of finish and freedom from scratches by buffing them.

In order to provide information as to the effect of heat treatment upon the corrosion-resisting properties, certain specimens were supplied heat treated, in addition to the specimens in their untreated condition, as rolled. The best resistance to corrosion is obtained with a quenching and tempering treatment, and therefore it was considered useful to include specimens which had received such treatment. For steel intended for structural purposes, however, this would be an expensive, if not an impracticable, procedure, and consequently it was thought desirable to include specimens which had received a simple annealing treatment.

Thus the different physical conditions of the specimens comprised :—

(1) As rolled.

(2) Annealed by heating to 800° C. and cooling slowly in furnace.

(3) Quenched and tempered by heating to 900° C., followed by a quenching in water, and then reheating to 400° C. and again cooling in water.

The series of eighteen specimens was finally arranged as shown in Appendix II, Table I, each set of three representing a particular combination of treatment and condition of surface. To each of these combinations the distinctive classification-letters were given, as shown in the Table, and to provide for these the scheme of code markings by drilled holes at the ends of the bars was extended. The specimens were finally completed and dispatched in March, 1924.

Mechanical Tests and Effect of Heat-Treatment.—As in the case of the original materials, mechanical tests were carried out on bars representing each of the three different physical conditions. These tests were made in the Hadfield research laboratories in exactly the same manner as those representing the main series of materials.

The test data obtained will be found included under the letters "S," "T," "V," "W" and "X" in the appropriate portions of Appendix II, Table III, where they are shown in relation to those of the other alloy steels, the remaining ferrous materials comprised in the complete research being given in Appendix II, Tables IV, V and VI. It will be seen that, in its rolled condition, this softer-grade material has a tenacity of 68 tons per square inch, and a Brinell hardness number of 329/341, as compared with a number of 600 for the harder-grade material. For this reason the bars of the harder grade were softened, for the purposes of the research, to a tenacity of 65 tons per square inch and to a Brinell hardness number of 300.

The effect of annealing has been to reduce considerably the hardness and tenacity of the material, and at the same time to increase its ductility and toughness under shock, while by the quenching and tempering treatment the hardness and tenacity have been increased without detriment to the ductility and toughness found in the material as rolled.

Under the quenching and tempering treatment employed with the harder grade of material, a tenacity of 50·2 tons and Brinell hardness number of about 220 were obtained with a ductility of 16 per cent., the softer grade having 81 tons tenacity, a Brinell hardness number of 400, and a ductility of 10 per cent. A temperature of 750° C. was, however, employed to temper the hard-grade material, as compared with 400° C. for the soft-grade specimens. In Appendix II, Table VII, are shown the detailed figures obtained for the elongation along the whole length of the tensile bars representing the three separate physical conditions. These figures are

thus comparable with those contained in Tables V, VI, and VII of the previous Paper.[1] Photomicrographs of the steel in its three different physical conditions are shown in *Figs. 1, 2* and *3*. These are taken on longitudinal and transverse sections in the same manner as for the materials of the main research, and at the same magnifications of 100 and 600 respectively. Reference should also be made to Appendix II, Table VIII.

Newer Types of Corrosion-Resisting Steels.—Still more recently, metallurgical research has resulted in the development of non-rusting steels which contain, besides from 10 to 25 per cent. of chromium, from 7 to 40 per cent. of nickel. In several ways these steels more nearly approach the ideal of a non-corrodible material than the chromium steels do. Among their more important advantages as compared with chromium steels is that they are resistant to a much greater range of corrosive agencies, while they also display improved non-corroding qualities, even in the absence of heat-treatment. Further, the method of preparation of their surfaces has not such an important influence on their resistance to corrosion.

The Committee gave due consideration to the question of including representatives of these new steels in their research, but concluded that, whatever their possible merits, the extent of their use in marine structures was likely, for the present, to be limited. In the meantime, therefore, the considerable extra expense of providing the necessary specimens was not likely to be justified by the value of the results obtained.

A record of some tests carried out by the Authors, in collaboration with the United British Oilfields of Trinidad, Ltd., which included steels of this type, is given later in this Paper.

Statistical Analysis of the Five-Year Results obtained in the Research of the Committee.

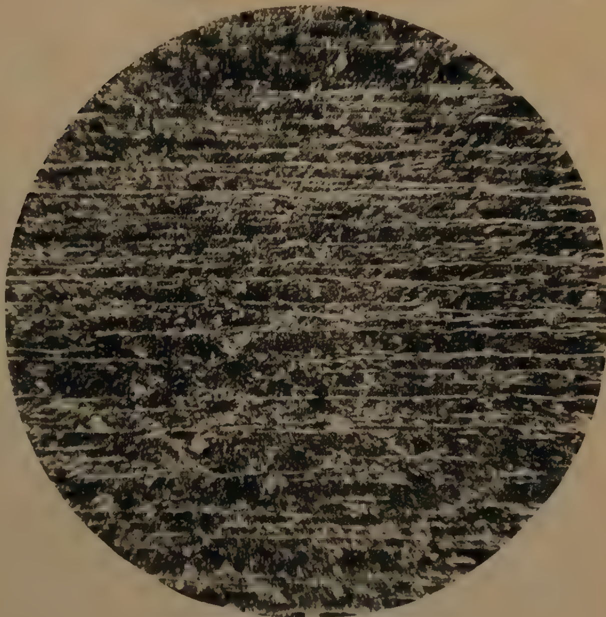
The varying effect of exposure under either aerial, half-tide, or total immersion conditions, and of the local conditions at the different ports where the various materials were exposed, makes it difficult, from an ordinary examination of the results, to assess their relative merits in numerical terms. The systematic planning of the research, however, makes it possible to group the data in various ways, and to examine them statistically. By this means it is possible not only to obtain an average figure for the relative performances of two materials or two types of material in any particular comparison desired, but also to trace conveniently the changes in the relative

[1] See footnote 1 on p. 4.

Figs. 1.

SECTION VI. CHROMIUM STEEL—SOFT GRADE. AS ROLLED.
(For Analysis see Appendix II, Table II)

Ref. No.
5250



LONGITUDINAL SECTION. X 100

Ref. No.
5251

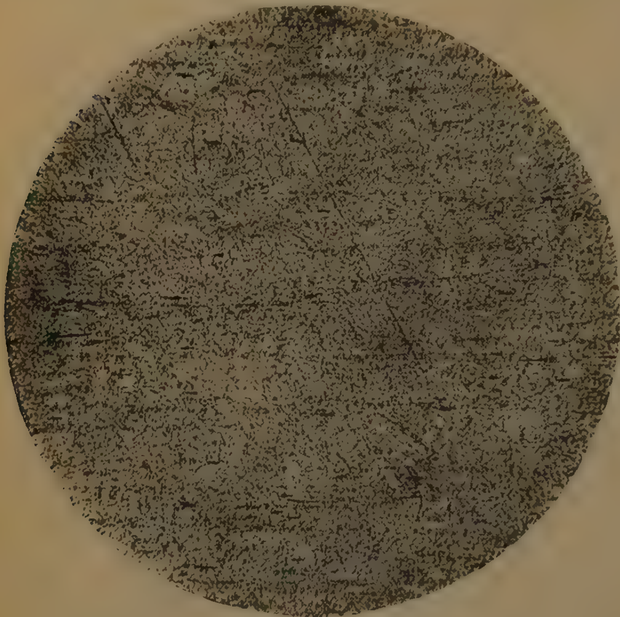
TRANSVERSE SECTION. X 600

4989.

Figs. 2.

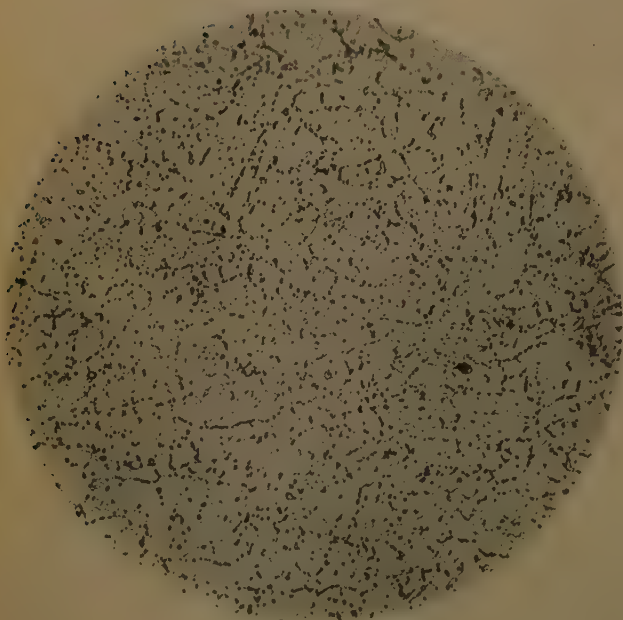
SECTION VI. CHROMIUM STEEL—SOFT GRADE. AS ANNEALED. 800°C. COOLED IN FURNACE. (For Analysis see Appendix II, Table II).

Ref. No. 3841



LONGITUDINAL SECTION. X 100

Ref. No. 3842



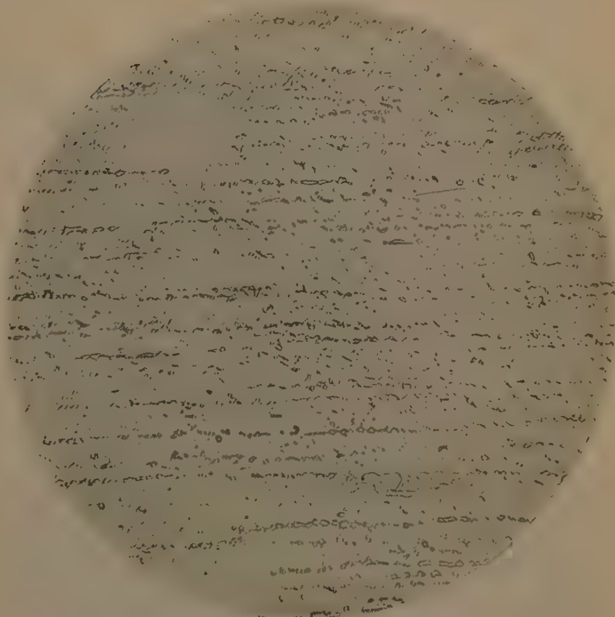
TRANSVERSE SECTION. X 600

Figs. 3.

SECTION VI. CHROMIUM STEEL—SOFT GRADE, AS QUENCHED AND TEMPERED. 900°C. WATER + 400°C. WATER.

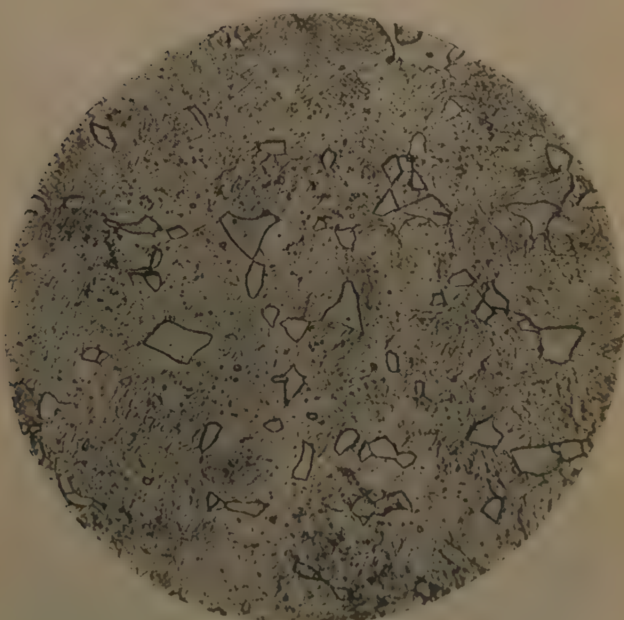
(For Analysis see Appendix II, Table II).

Ref. No. 3843



LONGITUDINAL SECTION. X 100

Ref. No. 3844



TRANSVERSE SECTION. X 600

4989.

Figs. 9.

SPECIMENS OF COPPER-CHROMIUM STEEL AND MILD STEEL AFTER EXPOSURE FOR ONE MONTH IN THE RIVER TEES AT MEAN TIDE LEVEL. THE CORROSION-PRODUCTS HAVE BEEN REMOVED.

MILD STEEL.

COPPER-CHROMIUM STEEL.

performances of the materials under changing circumstances. This method has therefore been employed by the Authors, and the results are as shown later. The method has the further advantage of enabling the relative corrosive influences of the different methods of exposure to be assessed numerically, as well as those of the different locations, namely, Halifax, Auckland, Plymouth and Colombo, where the exposure took place.

In the various statistical comparisons, the data both for the general wastage, namely, the loss in weight of the specimens, and for the pitting, as gauged by the depth of the deepest pit, have each been considered separately. The figures for general wastage, shown in the Committee's reports in terms of grams per 1000 square centimetres of surface, have been converted to an equivalent thickness of steel, expressed in millimetres, according to the following formula :—

Thickness in millimetres of steel removed

$$= \frac{\text{grams lost per 1,000 square centimetres}}{7{\cdot}8 \times 100}.$$

A uniform specific gravity of 7·8 for all the materials has been assumed; the error involved by the use of this assumption is comparatively small, and may be regarded as unimportant for the purpose for which the data are used. In general, the corrosion is not distributed evenly, and the thickness figure represents an average, in the same way that the figure for the loss in grams per 1,000 square centimetres takes no account of its distribution. Where pitting is of a pronounced and local character, the material removed from the pits may form a considerable proportion of the whole loss, and therefore the thickness calculated from the loss-figure will in such cases overestimate the average thickness of metal which is lost in a more uniform way from the general surface. Where, however, the pitting is of such a pronounced character, as for example in the case of 13-per-cent. chromium steel under certain conditions, this fact is sufficient in itself to decide the merits of the case.

The data have been translated into terms of thickness because engineers will probably find it easier to appreciate the amount of corrosion by this means. Again, being in the same terms, the relative extent of the pitting in relation to the general corrosion is made more apparent. The use which is made of the data on pitting in averaging the results can hardly be defended on strict mathematical grounds. Here again, however, as no stress is laid on individual figures or on small differences in the averages thus obtained for different materials, no serious misconceptions can arise, and on the other hand the method does enable comparisons of the

propensities for pitting to be made which would otherwise be difficult. The examination made is of the 5-year results, and only as regards the plain bars exposed singly, and under marine conditions.

It should be made clear that the analysis which follows later in this Paper is essentially of a broad nature, and is only intended to supplement the fuller reports prepared by the Committee. It obviously cannot take in account the more intimate details, especially as regards differences in the character of the corrosion in various cases. These details are, nevertheless, of some importance in a complete assessment of the merits of the different materials and of the influence of various factors, and are more fully studied in the Committee's reports.

Influence of Rolling-Mill Scale.—Specimens "A" and "C" have been specially provided for examining this influence, these specimens being from identically the same carbon steels as "E" and "F" respectively, but having had their scale removed before exposure.

General Wastage.—Appendix III, Table IX, shows that there is nothing very consistent in the way in which the previous removal of scale affects the general wastage which the steel suffers, although the evidence provided by the two different steels is nearly always in agreement under any particular conditions. In most cases, taking into account all the twelve methods and locations of marine exposure, the removal of the scale previous to exposure has the effect of increasing the amount of the general corrosion. Steel with its scale left on has the advantage in fourteen cases out of twenty-four.

Further examination shows that this superiority is mainly shown where the materials are totally immersed, the average being 0·489 millimetres as compared with 0·567 millimetres. Under aerial conditions (five cases) it is the scaled steel which shows less wastage, averaging 0·680 millimetres against 0·797 millimetres. Under half-tide conditions, the numbers are even—four cases each—although the steel with the scale left on has the lower average loss of 0·566 millimetres against 0·603 millimetres. The beneficial effect of removing the scale under aerial conditions, which is particularly marked at Plymouth, sufficiently exceeds its inferiority in the remaining cases to wipe out the adverse balance in the general average, which is practically the same in each case, namely, 0·618 millimetres (0·024 inches).

Pitting.—The important effect of removing the scale is, however, in regard to pitting. Against this particularly obnoxious form of corrosion its influence is almost entirely favourable, and in the majority of cases when the scale is removed the reduction in pitting

is striking. As a general average, the depth of the deepest pit is reduced from 2·14 millimetres to 0·92 millimetres.

As shown in Appendix III, Table X, out of the twenty-four cases where both scaled specimens and specimens of the same material bearing their scale have been exposed under parallel conditions, there are only two in which the pitting has been deeper on the scaled bars. These two cases are both with aerial exposure, and in one of them, at Halifax, the pitting is, as usual under aerial conditions, not serious. The other case occurred at Colombo, and the disparity found there is nothing like so great as in those many contrary cases where the pitting on the bar exposed with its scale on is deeper than on the scaled bar.

The results from the complete set of tensile bars with ground surfaces which were totally immersed at Plymouth, and which were given in Table IX of the Ninth Interim Report of the Committee, show that the effects of previously removing the scale of totally-immersed bars, as ascertained above, are fully confirmed for the whole of the ordinary and special steels, with the exception of the chromium steel "J." That is, their general wastage is increased with reduced pitting. With the chromium steel, and also with the cast irons, the general wastage is similarly increased by previous removal of the scale, but pitting is accentuated. The influence of rolling-mill scale on chromium steel is referred to further on p. 44.

Electrolytic action between the rolling-mill scale and the steel is no doubt responsible for the effects as regards pitting; when, however, the scale has entirely corroded away, as it has in practically all cases within the 5-year period, this action is no longer operative. When this stage arrives, the bars which have not been scaled, except that they have already acquired deeper pits, should be on much the same footing as those bars from which the scale has previously been removed. It should be interesting, therefore, to observe the further progress of corrosion as ascertained in the results of the 10- and 15-year periods of exposure on duplicate and triplicate bars.

The protective influence of the rolling-mill scale in reducing general corrosion is not readily accountable. Cases are mentioned on p. 52 where dissimilar ferrous materials in contact suffer less total corrosion than when they are separately exposed. It may be that the explanation in those cases would be similarly applicable in the present instance, which may be regarded as a similar combination but consisting of steel and rolling-mill scale.

Influence of Climate and Method of Exposure. (a) *Influence of Climate.*—The study of these factors is made here more particularly with reference to the ferrous materials which are commonly used,

namely, the rolled irons, cast irons, and ordinary steels. Such differences as the special steels display in response to their influence will be considered later when studying the merits of those materials.

Consideration of the influence of climate in promoting corrosion of ferrous metals, as shown in this research, cannot be made without taking into account the method of exposure. The effect of climate, as seen in Appendix III, Tables IX to XVIII, varies according to the way in which the steel is exposed, namely, aerially, at half-tide, or continuously immersed, and the effect of these conditions depends in turn on the climate.

(1) *In Air.*—The order of increasing severity as regards general wastage is always Halifax, Auckland, Plymouth, Colombo, whether the material concerned is a rolled iron, ordinary steel scaled or bearing its scale, or one of the special steels. It is remarkable in fact how this same order is preserved for every one, without exception, of the fourteen materials in these categories exposed at the four stations, representing fifty-six different specimens. In the case of the cast irons "Q" and "R" (Appendix III, Table XVIII), the corrosion in air at Halifax, Auckland and Plymouth has hardly gone far enough in 5 years to discriminate, but, consistently with the rolled products, the corrosion at Colombo is decidedly the greatest. The corrosion of the 36-per-cent. nickel steel "L" is almost negligible in air at all the stations, but among the small figures representing the losses Colombo is again actually the highest. Table XI (Appendix III) shows that the corrosion at Colombo is, for the rolled irons as a class, about eleven and a half times that at Halifax, and for the ordinary steels about fourteen and a half times, so that climate as experienced in different parts of the world has a very considerable influence on aerial corrosion. Combining the average rolled iron with the average ordinary steel, the corrosion losses are :—

Halifax	0·140 millimetres
Auckland	0·300 ,,
Plymouth	0·951 ,,
Colombo	1·834 ,,

The influence of climate on pitting under aerial conditions is not quite so consistent (Appendix III, Table XII); again combining the figures for average rolled iron with those for the average ordinary steel, the order in increasing severity is Halifax, Plymouth, Auckland and Colombo.

This order is shown also by the ordinary steels as a group, although for the steels "A" and "C," without scale, the relative places of Plymouth and Auckland are changed. For the rolled irons, which were exposed in a similarly scaled condition, it is the relative positions

of Auckland and Colombo which are reversed. It is characteristic of the cast irons that they show no pitting in these 5-year tests.

The average depth of the maximum pitting, as between the rolled irons and steels, is :—

Halifax	0·16 millimetres
Plymouth	0·68 ,,
Auckland	1·53 ,,
Colombo	1·82 ,,

Thus, in a general way, the tendency of the different climates to cause pitting is broadly the same as their tendency to cause general corrosion. Halifax has the most favourable climate in both respects, and Colombo usually has the worst. The relative positions of the effects of Plymouth and Auckland are changed, the former, at which much more general corrosion occurs, being much less active in causing pitting.

Clearly, climatic temperature, which is highest at Colombo and lowest at Halifax, and intermediate at Plymouth and Auckland, must have a large influence in promoting corrosion of both types. Other factors are also, no doubt, of importance, and probably it is the greater subjection of the specimens to sea-spray at Plymouth which is responsible for the general corrosion there being greater than at the other temperate location, Auckland, where the temperature is higher, although pitting follows the order of temperatures.

(2) *Under Half-Tide Conditions.*—Combining the figures for average ordinary steels and ordinary iron, the loss by wastage is found to be as follows :—

Auckland	0·186 millimetres
Halifax	0·388 ,,
Plymouth	0·418 ,,
Colombo	1·014 ,,

This order also applies to each of the rolled irons individually, and, as will be seen later, to all the special steels. For the ordinary steels there is a reversal in the positions of Plymouth and Halifax. Again, individually the scaled steels " A " and " C " and the cast irons preserve the same order as that shown by all the materials under aerial conditions. Thus, while the response of the various materials to the conditions at the different locations is not so completely consistent as for aerial exposure, the unanimity of the behaviour inside each group is striking ; Colombo, it will be noted, is always the most severe on every material.

As regards pitting, the relative severity of the different locations, taken as the average of the rolled irons and ordinary steels in the same way, is :—

Auckland	0·34 millimetres
Plymouth	1·31 ,,
Colombo	2·84 ,,
Halifax	3·36 ,,

As in the case of aerial exposure, the order is by no means consistent for the different materials, though the pitting at Auckland is least in all cases, and Plymouth always occupies a middle position. A striking feature is the extraordinary severity of the pitting at Halifax, which gives the highest figures for five out of seven materials, namely, two of the four steels and each of the three irons. Reference to Table X (Appendix III) shows that the steels which have had their scale removed behave in a similar way. At Colombo the steels are pitted in an excessive manner, but not so the irons. Here the scaled steels (Table X) behave like the irons, the behaviour of which therefore seems to be explained by their being similarly scaled.

(3) *Total Immersion.*—For ordinary steels and irons totally immersed, the average loss by general wastage, calculated in the same way as before, is :—

Auckland	0·430	millimetres
Plymouth	0·478	,,
Colombo	0·495	,,
Halifax	0·513	,,

It will be noticed that, in comparison with the order obtaining under half-tide conditions, Halifax goes two further places down the scale, to the lowest position. As may be expected where individual figures vary over such a small range, there is no apparent regularity in the response of the separate materials to the conditions obtaining at the different stations. For the irons as a group, Halifax causes the greatest corrosion, but for the steels Colombo is the most severe in seven cases out of eleven, this applying also to the steels which have been cleaned.

The effect in causing pitting is shown by the following average figures, the cast irons again being omitted as showing no discrimination between the various ports.

Halifax	1·04	millimetres
Plymouth	1·72	,,
Auckland	2·22	,,
Colombo	3·75	,,

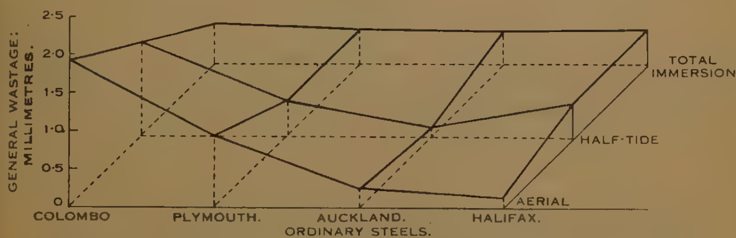
The order has reverted to that obtaining with aerial exposure, but here, as is the case with the general corrosion, the relative response of the different materials at the different locations is very variable, Colombo, however, giving the worst results (or equal to the worst) in seven cases out of nine.

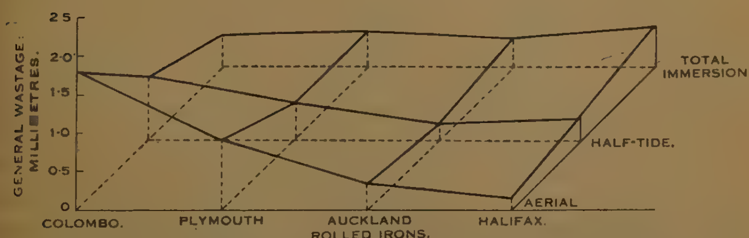
(b) *Influence of Method of Exposure.*—The above examination of

the influence of climate has already given indications of the effect of the method of exposure on the corrosion experienced.

General Wastage.—It is instructive to present the broad data in the form of diagrams, as shown in *Figs. 4*, *5* (p. 26) and *6* (p. 27). These clearly show, as far as rolled irons and ordinary steels are concerned, that as they become more and more subject to the effects of contact with the sea, from aerial exposure through half-tide conditions to total immersion, the effects of climatic

Figs. 4.



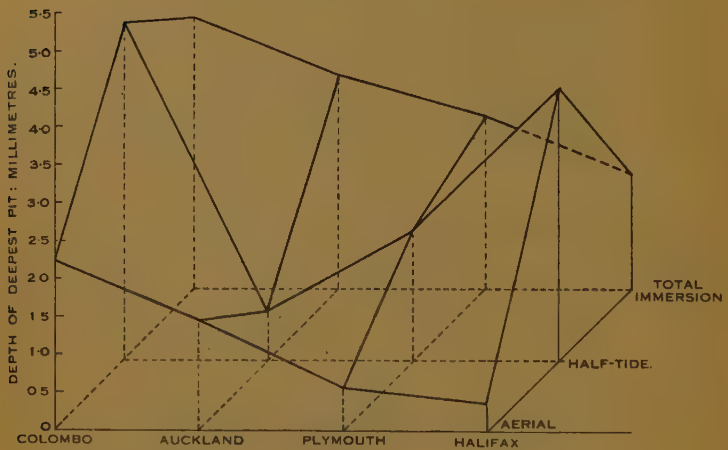


INFLUENCE OF CLIMATE AND CONDITIONS OF EXPOSURE ON THE GENERAL WASTAGE OF THE ORDINARY STEELS AND ROLLED IRONS.

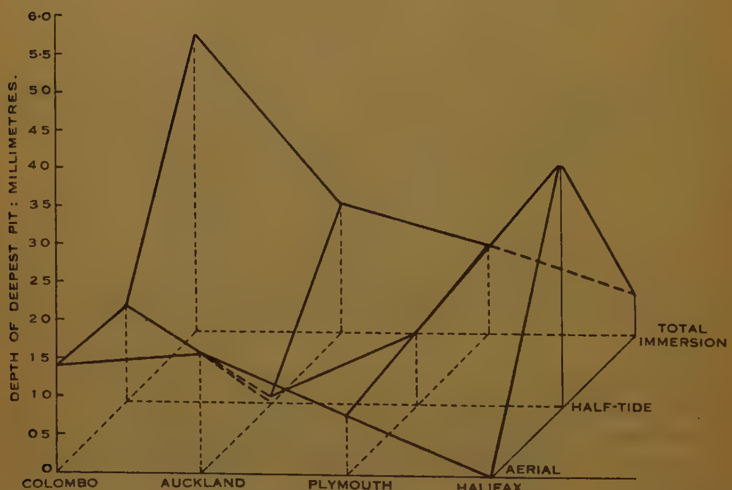
temperature in inducing corrosion become less and less pronounced. In the case of the irons as a group, it is noticeable, in fact, that in the hottest climate, Colombo, the losses with total immersion are actually not the greatest, and this is true for each of the irons individually.

On the whole the corrosion under total-immersion conditions is remarkably uniform in amount for each material. The severe extent of the corrosion which can be experienced with aerial exposure in hot climates is also apparent, and the wastage is still very considerable even when the steel or iron is intermittently immersed by the sea.

Figs. 5.



ORDINARY STEELS.



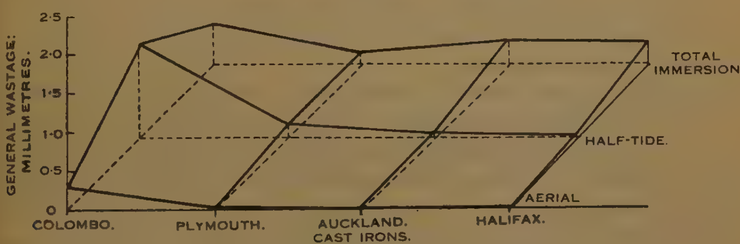
ROLLED IRONS.

INFLUENCE OF CLIMATE AND CONDITIONS OF EXPOSURE ON THE PITTING OF THE ORDINARY STEELS AND ROLLED IRONS.

The behaviour under half-tide conditions should especially be noted, particularly the fact that at any location where not intermediate in its effects between aerial exposure and total immersion, the corrosion at half-tide is actually less than for either of these conditions.

Pitting.—It is shown in *Figs. 5* that the influence of the conditions of exposure in promoting pitting is very different in several respects from their relative effects in causing general corrosion. The amount of pitting does not follow any such systematic relationship with the locations or methods of exposure, and to obtain the most schematic arrangement possible, it has been necessary to reverse the order of the two temperate locations, Auckland and Plymouth, from that used in the diagrams showing general wastage. Although there is apparently nothing very systematic in the response to climate

Fig. 6.



Influence of Climate and Conditions of Exposure on the General Wastage of the Cast Irons.

and to conditions of exposure, this is obviously not because of any fortuitous circumstances in obtaining the observations. The behaviours of the irons and steels bear, in fact, many points of similarity; they only differ to any great extent in their response to the aerial and half-tide conditions at Colombo. Bearing this in mind, the features seen are that, contrary to what happens with general wastage, pitting is greater for total immersion than in air at each location. Thus, at Halifax both the general corrosion and the pitting are increased with the change from the air to the sea, and at Colombo the general corrosion is much reduced and the pitting greatly increased. Under half-tide conditions, the extent of the pitting at Halifax is most extraordinary, in view of the very moderate general corrosion which occurs. Pitting under these conditions seems in fact to be taking a reverse order to that operating under aerial and total-immersion conditions, were it not for the fact that at Colombo pitting again becomes severe.

At this point, concordance in the behaviour of the irons and steels seems to end, and in other respects their response to varying conditions is peculiar to the two types of material. It should be noted, however, referring to Appendix III, Table IX, that the relatively moderate pitting experienced by the irons at Colombo is shared by the steels when they are similarly stripped of their scale. Under aerial conditions, and for total immersion, the pitting of the steels increases, on the average, in the order of Halifax, Plymouth, Auckland and Colombo. That is, the pitting is most severe in the hottest climate and least in the coldest, but the irons, on the other hand, suffer less pitting at Colombo than at Auckland. This contrast in the behaviour of the irons and steels is intensified with half-tide conditions at Colombo. Here the irons suffer only a moderate pitting, while that experienced by the steels is greater than under any other conditions.

Conclusions on the Effect of Climate and Method of Exposure.—The effects of these two factors are often so different on rolled irons and steels on the one hand, and cast irons on the other, that it is necessary to consider them separately.

Rolled Irons and Ordinary Steels.—The effects are not quite the same for each of these types, but they are similar in most respects. It is clear that, in the wastage by corrosion of iron and steel under marine conditions, the temperature to which the material is subjected is an important factor. At the same time, where there is actual contact with the sea, it would appear also that the local character of the sea-water itself must have some influence. Where the steel is kept totally immersed, the effects of climatic temperature are naturally at a minimum, and it is not surprising to find, therefore, a considerable degree of uniformity in the corrosion of any particular steel or iron for all locations from the hottest to the coldest. The fact that, in the case of the rolled irons, the wastage at Colombo, where the sea temperature (77/86° F.) is distinctly higher than at the other locations, is by no means the greatest, is an indication that the character of the sea-water may be of more importance in its effect on such materials than its temperature. Different materials, as, for example, the rolled irons and the ordinary steels taken as distinct classes, respond differently to changes in the character of the sea water and in its temperature, as will be noted later in discussing the relative merits of such materials.

In air, where the effects of climatic temperature have full play, this appears to have almost a predominating influence. In the coldest climate, Halifax, the general wastage in air is less than when the steel is totally immersed. Its amount increases through the more temperate climates of Auckland and Plymouth, and at the

hottest, Colombo, the corrosion greatly exceeds that experienced in the sea. At the temperate locations, Plymouth and Auckland, where the sea temperature is nearest to that of the air, the corrosion is not greatly different in air and in the sea.

The evidence as to the behaviour of iron and steel under half-tide conditions is especially instructive, contrary as it is to the widely accepted idea that exposure " between wind and water " is particularly favourable to corrosion. In the large majority of cases among ordinary steels and rolled irons, the wastage at half-tide is intermediate between that experienced under aerial and total-immersion conditions. Where this is not the case, in all but one instance (steel " B " at Halifax) it is less than under either condition of exposure. In practically every case, the special steels included in this research, which have played no part in the experience on which the erroneous impression mentioned has been based, all suffer an intermediate amount of corrosion under half-tide conditions. The only results in the Committee's research which justify the general impression are those for the cast irons at Colombo as referred to later.

At the same time, the influence of half-tide conditions in promoting corrosion would appear to contain some complicating factors. At Plymouth and Auckland, the corrosion of the rolled irons and steels is usually less than with either aerial exposure or total immersion, and at Auckland it is particularly small. It is not clear what causes this difference; a feature which these two ports have in common is a higher tidal rise and fall than is the case with the other two, but it is not easy to see how this can explain the facts mentioned, except that they may be due to the decreased time during which only partial exposure takes place under such conditions.

One fact which may help to explain the effects of half-tide conditions is that Halifax progresses steadily in the order of its severity among the Committee's stations, from being the least corrosive under aerial conditions, to the second position at half-tide and greatest for total immersion conditions. It would appear that there must be some particularly unfavourable characteristic in the water at Halifax, or possibly in the nature of the molluscs there as affecting the access of the sea-water, which operates to a greater degree as its contact with the iron or steel becomes more continuous. Auckland, however, which improves its relative position when there is contact of the iron or steel with water, apparently has some favourable feature in its sea-water tending to render it non-conducive to corrosion.

The effects of climate and methods of exposure in promoting pitting show such extraordinary anomalies that it is difficult to

establish any definite relationship ; the factors which cause corrosive action to assume the form of local attack are clearly affected in a different way by changes of temperature or character of the sea-water from those factors which decide the amount of general wastage. It can be said, however, that complete immersion in the sea definitely favours pitting as compared with aerial exposure, whether the general wastage is greater under water, as at Halifax, or less, as at Colombo.

Sea-water is naturally a more efficient electrolyte than any moisture which collects on specimens exposed to the air, and, so far as pitting is caused by electrolytic action of the original scale on the specimen, this fact is probably the main explanation. The behaviour of specimens previously stripped of their scale indicates, however, that this cannot be the whole cause. The exceptions to the general rule that pitting is deeper under conditions of total immersion are certainly more numerous with the previously-cleaned materials, and, on the general average, pitting is very little deeper for the cleaned steels " A " and " C," Table X, Appendix III, than for the corresponding steels " E " and " F " with their scale still on, the figures being 0·93 millimetre and 0·73 millimetre respectively. For the rolled irons which were similarly scaled, the depth of pitting for total immersion is about twice that for aerial conditions. In the present state of knowledge, and without experimental research on this matter, it does not seem possible to go further than this in the way of suggesting an explanation, except that the presence of an adequate amount of moisture appears to be an essential in all theories regarding pitting. This is not always assured in air.

Increased temperature in these underwater conditions appears to play a prominent part in promoting pitting. Under aerial conditions, the moderate amount of pitting experienced seems to respond generally to climatic temperature. The variations of temperature between the various stations is much greater in the air than in water, yet the influence of temperature on pitting would appear, from the figures, to be much less in the air than in the water. In the case of the irons, Colombo does not always show the greatest pitting, although it does for the steels, and in nearly all cases, both with steels and irons, Halifax shows the least.

As in the case of the general wastage, the study of half-tide conditions, as regards pitting, reveals many complications. At Auckland, where the general corrosion is seen to be remarkably small, it is also of a marked non-pitting character. This raises the question as to how far the depth of pitting may be related to the amount of general corrosion. If there is a relationship, the small

amount of pitting at Auckland would be a natural consequence of the small extent of the corrosion generally. The Committee's results, however, do not show any such systematic connection, there being many cases where the general corrosion is small and the pitting deep, and on the other hand, many where the reverse is the case. Here, again, a comparative study of the 10-year results may throw some further light on the question.

The excessive pitting at Halifax is certainly curious, and has already received attention; examination of the general wastage figures has suggested some specially corrosive feature in the sea-water itself, or a difference in the character of the molluscs at Halifax. This does not, however, seem adequate to explain the present fact, especially as no corresponding effect is seen under conditions of total immersion. It is possible that the explanation is due to the oil which contaminates the waters of this port; certainly such contamination could only affect the bars exposed at half-tide. If this solution were proved correct, however, it would not be sufficient to explain the results for total immersion. The pitting at Colombo is, for the steels, greater than under aerial or total immersion conditions, and for the irons less than either. This relative behaviour is not reflected in the results for total immersion.

Cast Irons.—The wastage of the cast irons is, under similar conditions, generally very much less than that of the rolled irons and steels, and the pitting that they experience is negligible in a 5-year test. Disregarding, however, the relative merits of the cast irons, and considering only the points of difference or similarity in their response to climatic conditions and method of exposure, the features which emerge are set out below.

Temperature, however, does not have a controlling influence on the corrosion experienced under conditions of total immersion. In the particularly warm water at Colombo, the corrosion is definitely greater than at the other locations, and the cast irons behaved like the ordinary steels rather than like the rolled irons. There is, however, still no very considerable variation experienced in the corrosion at the different locations.

The corrosion under half-tide conditions is more nearly intermediate between that for aerial exposure, which is practically nil, and that for total immersion. There is a marked exception. however, in the case of Colombo, where the corrosion of the cast irons assumes considerable proportions, equivalent to 1·25 millimetre.

Summary.—To sum up, the varying degrees of corrosion observed for the different ferrous materials in general use, under the different

conditions to which they may be subjected in marine structures, appear to be the result of a balance of factors as follows :—

(1) The material itself.
(2) Temperature, the range of which in different climates is much less under the tempering influence of contact with the sea than where the iron or steel is in the air.
(3) Actual contact or otherwise with the sea-water, and the continuity or otherwise of the contact.
(4) The character of the sea-water and of the molluscs adhering to the material.

The probable effect of these factors, which are often different as regards the amount of the corrosion they cause with different materials, and the character of corrosion, namely pitting or non-pitting, have, so far as they are ascertainable from the data, been examined in detail above. With equal ranges of temperature, there does not appear to be much difference in the amount of corrosion in the sea and in the air; sea-water, however, promotes pitting more than air.

There is evidence that the character of the sea-water plays a prominent part, but, in the absence of sufficient data as to its constitution and condition at the different locations, it has not been possible to trace a definite connection. It appears to be more important in its effects on the character of the corrosion, namely, pitting, than on the actual amount of corrosion.

Relative Merits of the Different Materials in Resisting Corrosion.—The various materials will be considered first in their classes, and then individually. Although the materials naturally occupy different positions in the frames, an examination of the actual results indicates that this has had no appreciable influence on the comparisons made of the behaviour. It should be recognized that the true assessment of relative merits will depend to some extent upon the form in which the materials are to be used. Thus the life of sheets and thin plates is largely determined by their becoming perforated. In such a case, the question of pitting must be given special consideration. In several cases, at the rate at which pitting is progressing, it is possible that some definite discrimination, on the basis of actual perforation of these ½-inch plates, may be feasible at the end of 15 years. Meanwhile, the pitting which has actually taken place may be taken as the best indication possible.

On the other hand, where, in practice, sections are substantial and unlikely to be perforated where pitting is not abnormal, loss in weight is probably the best general guide.

Rolled Irons Compared with Ordinary Steels.—The average wastage in each of these classes, under the different conditions of exposure and climate, is given in Appendix III, Table XI. As regards total wastage, there is very little to choose between the irons and steels in this period of 5 years, and there does not seem to be anything systematic as to any particular set of conditions favouring either the irons or the steels. One characteristic of the irons in which they are different from the steels, and which is not explained by their cleaned condition, is that their wastage at Colombo in underwater conditions is not the greatest among the various locations. It has been concluded earlier that this may possibly be due to a greater response to the character of the sea-water, and a less response to its temperature, than in the case of the steels. Their relative behaviour as regards pitting is shown in Appendix III, Table XII.

Although the irons almost always show a definitely lesser tendency to pitting, it must be remembered that they have had the advantage of having had their scale removed. This superiority of the irons, however, is much less in every case than that of the clean steels over the steels with their mill-scale untouched. If scaling has the same relative effect on the irons, it would appear that under comparable conditions they may be more prone to pitting than the steels.

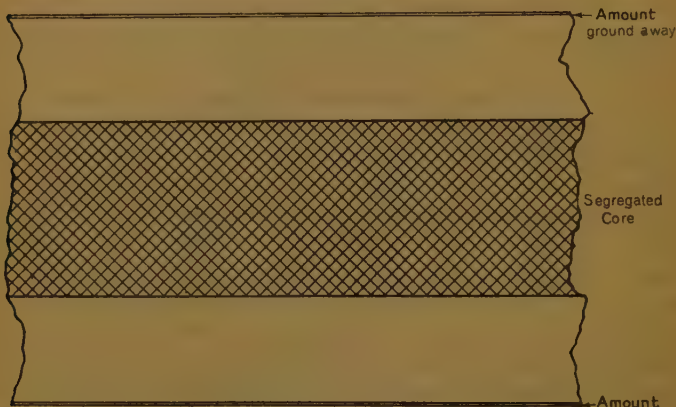
Rolled Irons Individually.—The separate wastage-figures for these are shown in Appendix III, Table XIII. It is a characteristic of ingot irons, of which the material " M " is representative, that bars rolled from them have an outer layer of comparatively pure material surrounding a less pure core. It should therefore be noted that the very small amount removed by grinding from the specimens previous to exposure was insufficient to reduce the thickness of the outer rim by more than a quite trifling amount, as shown by measurements made on bars held in reserve for the purpose of the research, and by *Figs. 7* (p. 34). Six bars with their scale on averaged in thickness 0·514 inch. The same bars, ground to remove the scale in exactly the same manner as those submitted to exposure tests, were found to average 0·509 inch. The average thickness removed therefore is 0·005 inch, or only 0·0025 inch per side. Thus it is quite impossible that the grinding operations could have influenced the results obtained from this material " M " by removing its pure outer rim.

This is further illustrated by the sketches in *Figs. 7*, which indicate the thickness of the rim, as determined by taking a sulphur-print of a transverse section of the bar. The amount removed by grinding can only be rendered visible on an enlarged scale as shown in *Figs. 7*. From Table XIII it will be seen that the relative behaviour of these

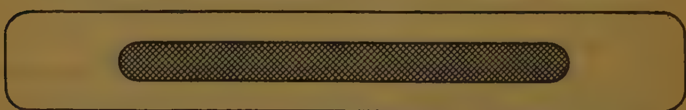
irons is very consistent. The ratio of the losses varies, but the same order of merit is retained in all except one of the varying circumstances.

The Low Moor wrought iron "N," on the results of general wastage, is 13 per cent. better than the next best, Swedish iron "P," but this superiority is less marked under conditions of total immersion. These conditions, as will be seen in other comparisons which will be made, tend to level out the differences between any two materials, at any rate as regards their general wastage. Aerial con-

Figs. 7.



(Four times actual size.)



(Actual Size.)

Amount Ground off the Ingot Iron Bars, "M," before Exposure, in Relation to the Segregated Core.

ditions are on the whole most consistently favourable to the Low Moor iron, which then shows an average advantage of 19 per cent. over the Swedish material. Among the various stations, Low Moor iron shows up most favourably at Plymouth, with an average advantage over Swedish iron of 23 per cent. under the three different conditions of exposure.

Pitting.—The results as regards pitting are given in Appendix III, Table XIV, and although, on the average, the Low Moor iron shows an advantage of 14 per cent. over the Swedish iron it cannot be said that there is any consistency in such behaviour. In ten cases out

of twelve where there is any superiority one way or the other, the Swedish material is actually the better in five of them. The Low Moor iron owes its better average to its pronounced superiority in the remaining five cases, amounting to as much as 62 per cent. for total immersion at Colombo, but in regard to pitting, for which again it is the best on the average, no particular station or method of exposure appears to be consistently favourable or otherwise to the Low Moor iron. On the whole, therefore, Low Moor iron is clearly the best in this class on both counts, namely, general wastage and pitting.

An examination of Appendix III, Tables XIII and XV, will show that the Low Moor iron is distinctly the best, as regards its low general wastage, of any of the ordinary rolled materials, whether irons or steels. It seems clear from Table IX (Appendix III) that it does not owe this fact to its having been scaled before exposure, as such scaling has not improved the steels on which it was tried. The choice of Low Moor iron by the Committee to act as a general standard of comparison was therefore well founded. The scaled steels pit to a rather less degree than the Low Moor iron, but all are quite satisfactory in this respect.

Ordinary Steels Individually.—The wastage figures for the ordinary steels, shown in the same way as for the rolled irons, are given in Table XV (Appendix III). Their order of merit, based on the results of general corrosion comprising the whole of the varying conditions of exposure, is rather surprising, the purest steel "E" showing the highest wastage (0·630 millimetre). The best steel is the one with the highest carbon, but of an intermediate purity in sulphur and phosphorus.

The range of quality is, however, only small, "E" being only 5 per cent. worse than "D," so that there is really little to choose, in an average way, between any of the steels from the point of view of the corrosion that they suffer, and it is not surprising in these circumstances that the order of merit varies widely in different cases. It is the location, rather than any particular method of exposure, which seems to discriminate between the different materials. Thus Halifax is particularly favourable to both "D" and "E," and gives inferior figures for "B"; at Colombo, on the other hand, "B" is actually the best and "E" the worst. Steel "E" also behaves particularly well at Plymouth, but in other cases the materials appear in almost every possible order, and small factors would easily alter their relative positions.

At the same time, taking an average of the four stations, it is interesting to note that the impure steel "B" improves its order of merit in passing from aerial conditions, where it is the worst, through

half-tide conditions where it is third, to total immersion, where it is actually the best.

Pitting.—The two steels "B" and "E," which, as regards general wastage, are the worst, come out, as regards pitting, slightly the best, as shown in Table XVI (Appendix III). Here again, however, the relative merit varies in different circumstances. An average superiority for "B" is shown under both aerial and total-immersion conditions, but under half-tide conditions the order of merit of the materials shows striking reversals as between the different stations, "B" being the best at Halifax and the worst at the other three stations. It is noteworthy that, of all the ordinary steels and wrought irons, this steel "B" with its rather high sulphur and phosphorus contents is the only one able to contend effectively with the specially energetic pitting action which occurs under half-tide conditions at Halifax.

On the whole, therefore, having regard to both general corrodibility and propensity to pitting, while it is possible, under any of the specific circumstances adopted for the Committee's research, to make the most suitable selection from the four steels, a choice could not be made with any certainty under a different set of conditions.

Cast Irons.—The cast irons show no pitting in these 5-year tests, which is a valuable feature and must be taken into account in comparison with the rolled irons and steels. The small losses which they sustain under most of the conditions make it almost immaterial whether they are compared with the rolled irons or with the ordinary steels, but the irons are chosen as giving the better comparison.

As a general average (Table XVII, Appendix III), the cast irons are about 58 per cent. better than the rolled irons, with a loss of 0·245 millimetre as compared with 0·579 millimetre. The different character of the response of the cast irons to climate and conditions of immersion, as compared with the rolled materials, has already been referred to, and is shown in *Figs. 4, 5* and *6*. The cast irons show to the best advantage under aerial conditions, their wastage being only 17 per cent. of that of the rolled irons at Colombo, where the rolled irons are attacked heavily. At the other stations, where the loss on the rolled irons is still considerable, that on the cast irons is hardly appreciable, except by the delicate means of weighing which was employed. Except at Colombo, the degree of superiority of the cast irons falls off in the order of aerial (where the losses are insignificant), half-tide (75 per cent.) and total-immersion (46 per cent.) conditions. Their particular susceptibility to attack in contact with the sea at Colombo makes them actually worse than the rolled irons under these particular conditions. Among the different stations,

Plymouth is most favourable to the cast irons, Colombo being the least favourable.

The choice of cast iron in preference to rolled iron or steel in portions of marine structures is naturally determined to a large extent by its greater suitability, from a purely constructional point of view, for particular members. The results obtained in this research may encourage a preference for cast iron on the grounds of its lesser corrodibility, and so far as aerial conditions are concerned, this would seem to be justified. The internal corrosion which the cast irons suffer in contact with the sea, and even under aerial conditions in a hot climate like Colombo, rather tends, however, to establish the preference in favour of rolled irons or steels where such conditions are concerned.

Cast Irons Individually.—Comparison in Table XVIII, Appendix III, of the merits of the hot-blast (" Q "), and cold-blast (" R ") irons is comparatively simple, the latter proving superior in a distinct and remarkably consistent way. Only when the losses on both materials are almost immeasurably small are the figures for " R " actually the greater.

In the general average the cold-blast iron is 14 per cent. better than the hot-blast material, but its superiority is greatest (40 per cent.) under aerial conditions, and least (only 7 per cent.) with total immersion. The cold-blast iron appears in about an equally favourable light at all the different locations.

Special Steels.—The steels comprising this group are of a varied character and show marked differences in their behaviour. No useful purpose would therefore be served by averaging the figures which they give, as was done in the case of the ordinary irons and steels. Such consistent characteristics as they have as a class will appear in the individual examination which follows of the various types, namely, copper, nickel and chromium steels.

(*a*) *Copper Steels.*—The carbon steel against which these steels should most logically be gauged is " F," which is nearest in its carbon-content, although " D " with 0·40 per cent. carbon comes nearest in manganese-, sulphur-, and phosphorus-contents. As has been seen, within the limits of composition of the carbon steels included in this research, there does not seem to be any definite connection between corrodibility and analysis. As " D " is, on the average, the best of the carbon steels as regards wastage, and better than " F " as regards pitting, it is probably the better choice as a standard with which to compare the copper steels.

As regards wastage, Table XIX, Appendix IV, shows that the copper steels are nearly always better than the best ordinary steel, the only case where they have failed to be so being under half-tide

conditions at Colombo. The improvement is also progressive with increasing copper-content, steel "G" with ½ per cent. of copper being 17 per cent., and steel "H" with 2 per cent. of copper 21 per cent., better than the ordinary steel on the general average figures. This is invariably the case under aerial conditions, but contact with the sea-water is less favourable to the 2-per-cent. copper steel. Under these conditions, ½ per cent. of copper gives the best results in the majority of cases, and an increase to 2 per cent. causes a deterioration, although the material still shows less wastage than ordinary steel.

Under aerial conditions, and in the most favourable case (Auckland), steel "H" shows an advantage of 74 per cent. over the ordinary steel. Under half-tide conditions, steel "G" shows a maximum advantage (at Halifax) of 55 per cent., and with total immersion an advantage of 27 per cent. (at Colombo). Among the various locations Colombo is the most unfavourable to copper steels, there being little to choose between the other stations.

Pitting.—As regards pitting, Table XX, Appendix IV, shows that the copper steels are subject to this form of corrosion to about the same extent as the plain carbon steels, with which pitting cannot be said as a rule to constitute a serious danger.

Steel "H," with 2 per cent. copper, is slightly superior to steel "G" with ½ per cent. copper, and shows this superiority under the majority of circumstances. Steel "H" is about equal to the best of the carbon steels, and "G" is about the same as "D," which is third best in its class. The relative merits of the copper steels in this respect vary with the circumstances. It will be noted that they do not overcome the peculiar conditions existing at Halifax for half-tide immersion, which result in such severe pitting.

On the whole, therefore, the addition of ½ per cent. or over of copper causes an appreciable improvement in the resistance of steel to marine corrosion. Where only aerial corrosion is concerned, 2 per cent. of copper may be added with advantage, but where there is contact with the sea nothing is gained in going beyond ½ per cent. For aerial conditions, however, copper does not improve steel to the extent of making it equal to cast iron in its resistance to corrosion.

It is useful to compare the results shown for copper steels in this research with what is known about their characteristics from other investigations. Most of such other work has been carried out with steels containing from 0·2 to 0·3 per cent. of copper, for the merits of which claims were being made in America as long ago as 1916.

One of the Authors, in his previous Paper to The Institution in

1922,[1] gave some account of the state of knowledge at that time, which on the whole was not very definite. His own tests had, however, indicated that such advantages as copper steel having these very low percentages would be likely to possess would be shown under conditions where sulphuric acid was active, either as the liquor itself, or in sulphur-laden atmospheres, as exist in urban and particularly in industrial districts. Its resistance to a clean atmosphere, or to fresh or salt water, showed little, if any, improvement over similar steel without a copper addition. Results pointing in the same direction had been obtained by the late Dr. Allerton S. Cushman and in an independent investigation carried out by the German ironmasters.

Much experience has since been obtained, both in this country and in the United States, with copper steels, which generally bears out the impressions then formed, and it has now been proved that the use of copper steel is beneficial against sulphurous conditions. Sir Henry Fowler, M. Inst. C.E., in 1922, recognizing its possibilities for dealing with the corrosion experienced in the smoke-boxes and ash-pans of locomotives, ordered from the Authors' firm some 5 tons of copper-steel plates, varying in thickness from ¼ inch to ½ inch, which were used for this purpose. The Authors have Sir Henry's authority for saying that their behaviour was so satisfactory that such copper steel has since been adopted generally for the smoke-boxes and ash-pans of the locomotives of the London, Midland and Scottish Railway. The analysis of this steel was :—

Carbon	0·17 per cent.
Silicon	0·10 ,,
Sulphur	0·023 ,,
Phosphorus	0·027 ,,
Manganese	0·43 ,,
Copper	0·39 ,,

At the East Hecla works of Messrs. Hadfields, Ltd., in Sheffield, the heavy annual expenditure for roofing sheets also prompted a trial of copper and other steels, which has been going on since March, 1927. This expenditure provides a remarkable instance of the serious effects of corrosion. At the works mentioned there are some 50 acres of buildings, most of them protected with steel roofing sheets; in the corrosive atmosphere of Sheffield these suffer quite severely, in fact the average quantity of sheets required during the years 1920 to 1933 amounted to no less than 84 tons per annum, not counting asbestos protected sheets. The maximum tonnage

[1] Footnote, p. 4.

purchased in one year was no less than 145 tons, and—as a reminiscence of the War period—in 1920 no less than £57 per ton was paid for these sheets, the present price being about £13 5*s*. per ton. The trials mentioned are still continuing, their duration having been protracted by the fact that in the earlier years the rate of corrosion was especially slow. This, it is significant to note, can certainly be attributed to the reduced activity in those years of the industries of Sheffield.

At the present time neither the copper-steel nor the ordinary mild-steel sheets included in the trials are apparently within measurable distance of their useful life. From present appearances, steel containing 0·35 per cent. copper is distinctly promising as compared with the usual mild steel. Steel with 0·22 per cent. copper is also being tested, but so far this quality does not appear to be behaving quite so well. Such measurements of the thickness of the sheets as are possible without disturbing them indicate that the 0·35-per-cent. copper steel is corroding at about 30 per cent. of the rate of the unalloyed sheets, and the steel with 0·22 per cent. of copper at about 33 per cent. The sheets used in these trials are galvanized, but not painted, and it is interesting to note that until the coating of zinc corroded away the superiority of the copper steel did not assert itself; thus the experience of the American expert, the late Dr. Allerton S. Cushman, who studied this question fully, appears to be confirmed. It is hoped to present the results of these trials to The Institution when they are completed.

The tests conducted by the Committee of the American Society for Testing Materials have been of a varied character, and at the time of writing were still continuing, many of the specimens having been under exposure for over 17 years. The report for the year 1932–33 shows that in certain atmospheres the life of steel sheets is actually improved by the addition of 0·2 to 0·3 per cent. of copper. On the other hand, in sea-water at various locations, such copper steels give little, if any, longer service than non-copper steels.

The specimens are all in the form of sheets, and their performance is ultimately gauged by the rate at which they are perforated by corrosion, so that their propensity to pitting plays an important part. This, as the character of the results obtained in the research of the Committee of The Institution has clearly shown, may put a different complexion on the relative values of different ferrous materials from that obtained where the criterion adopted is loss in weight. Nevertheless, it is a sound criterion where the use of sheets or thin plates is contemplated. In this country the advantages of low-percentage copper steel are being appreciated to such an extent that many thousands of tons are being supplied annually.

Dr. F. N. Speller[1] estimated that about 9 million tons had been produced in the world during the previous 20 years. Whether its limitations are fully understood, however, is not clear, and disappointment may result in some cases.

It seems, however, to be established that, for marine purposes or for other service where sulphurous conditions do not operate, little advantage is likely to be obtained from the use of steel with such a low content of copper as from 0·2 to 0·3 per cent. The choice in the Committee's research of steels with large percentages proves, therefore, to have been a wise one, seeing that definitely favourable results have been obtained with such larger percentages as 0·5 and 2·0. It would appear that the amount of copper which must be added to steel to give results of practical value must be varied according to the working conditions proposed. It does not seem to have been ascertained, however, whether even comparatively large additions would ever give results of value in an uncontaminated air such as is found in rural districts. For sulphurous conditions from 0·2 to 0·3 per cent. is ample, but this amount is insufficient to meet marine corrosion.

Nickel Steels.—The nickel steels comprise two, "K" with 0·31 per cent. carbon and 3·75 per cent. nickel, and "L" with 36 per cent. nickel. As with the copper steels, they are probably best compared with the best plain carbon steel "D." The relative figures obtained for the wastage show an excellent consistency, as set out in Table XXI, Appendix IV, the 3·75-per-cent. nickel steel being under all circumstances superior to the plain carbon steel, and the 36-per-cent. nickel steel still better. On the general average, "K" is 38 per cent. better than carbon steel, and "L" no less than 81 per cent. better. The degree of superiority varies with circumstances, being most marked under aerial conditions, and least for total immersion.

In air the corrosion of "K" averages about one-half that of ordinary steel, though it is actually 86 per cent. better at Plymouth, and shows only 20 per cent. superiority at Colombo. Under half-tide conditions it averages 37 per cent. better, and for total immersion only 25 per cent.

The results for the 36-per-cent. nickel steel are altogether excellent, and for aerial exposure quite remarkable. Under these conditions the corrosion it experiences is uniformly low, representing from about ½ per cent. to 3 per cent. of that for ordinary steel. Even at Colombo, where corrosion occurs to such a marked degree with ordinary irons and steels, the losses are almost insignificant, repre-

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[1] "Corrosion Problems of Iron and Steel," *Iron Age*, vol. 133 (1934), No. 24, pp. 28 and 68.

senting an average of only 0·0005 inch over the surface of the steel. Under half-tide conditions, the 36-per-cent. nickel steel is 80 per cent. superior to ordinary steel, and with total immersion it is still able to reduce the corrosion, as an average, by one-half.

The response of these nickel steels, both to climatic conditions and to methods of exposure, is interesting. The losses sustained by steel "L" in air, small as they are, seem to increase slightly in warmer climates, and the wastage of the 3·75-per-cent. nickel steel "K" is about fifty times as great at Colombo as it is at Halifax.

With total immersion, the losses for each steel are less at Colombo than at Halifax, so that the corrosion of nickel steel under these circumstances appears to be more dependent upon the character of the sea-water, and possibly upon the activities of the molluscs, than on temperature.

Pitting.—As regards pitting, there is one abnormal result: the "K" bar under half-tide conditions at Colombo was completely perforated. From appearances and observations made during the cleaning of the bars after exposure, this is believed to be due to the molluscs having been especially active; it is best disregarded, therefore, in the present connection, the actual extent of the pitting being in this case regarded as undetermined.

The figures in Appendix IV, Table XXII, show that, in six cases out of eleven, the pitting is in the decreasing order "D," "K," and "L," and that "L," with 36 per cent. of nickel, in ten cases out of twelve is far less prone to pitting than the ordinary steel "D." In one case, however, that of total immersion at Halifax, the order is reversed. On the average, the addition of 3·75 per cent. of nickel reduces the depth of the pitting by about one-quarter, and 36 per cent. of nickel reduces it by as much as two-thirds. The way in which these nickel steels overcome the tendency to the vigorous pitting which is associated with half-tide conditions at Halifax should be especially noted. The results for the 36-per-cent. nickel steel are remarkable; it not only reduces corrosion to an altogether insignificant amount, even under the severe conditions of tropical air, but it is able to reduce the wastage by one-half, even in underwater conditions. In this respect, it is better than the cast irons, which effect an improvement in wastage of 30 per cent. but suffer internal deterioration.

Various factors must be taken into account when considering the practical application of this high-nickel steel. Its cost, due to the large percentage of nickel, is considerable, but there might be some scope for its employment, wherever a long life will balance a high first cost. The mechanical characteristics of this material, which is of the type known as austenitic, differ somewhat from those of ordinary steel. One of its features is that the yield-point and

tenacity are determined to a considerable extent by the degree of cold work to which it has been subjected, and also, in the case of hot-rolled material, by the final temperature of rolling. By referring to Appendix II, Table V, it will be seen that its yield-point was 23·3 tons and its maximum stress was 38·9 tons per square inch. After heat treatment, which largely removed the hardening effect of rolling, these figures were reduced to 18·9 tons and 34·0 tons respectively. Its ductility and toughness were excellent, as shown by the figures in Table V, which relate to elongation and reduction in area.

Since many of the sections required for marine structural work are much heavier than these comparatively small test-specimens, their finishing temperatures in rolling would necessarily be rather high, which would adversely affect the yield-strength. Specimens forged at as high a temperature as 1,055° C. gave the following results :

Elastic limit (0·005 per cent. set) . . .	15 tons per square inch
Yield-point	20 ,, ,,
Maximum stress	37 ,, ,,
Elongation on 2-inch test length . . .	42 per cent.
Reduction in area at fracture	66 ,,

The Authors present these facts so that a correct judgment of the merits of this steel can be formed by those interested, who can also decide whether it might be possible, in certain instances, to take advantage of its excellent resistance to corrosion. It is also interesting to note that the stiffness and work-hardening characteristics associated in general with austenitic steels are only present in a moderate degree in this steel "L," containing 36 per cent. of nickel. The Brinell hardness number can, however, be increased, by cold working, from about 155 to 241.

A further point which should receive consideration is that, as the Committee's research has shown, where this steel or chromium steel are in contact with ordinary iron or steel the corrosion of the latter is greatly accelerated. In composite structures employing 36-per-cent. nickel steel in conjunction with other steels, provision would have to be made to prevent or guard against the effects of this electrolytic action, at any rate for underwater structures. In all probability it would be necessary, for the same reason, to adopt a special steel for the rivets and bolts, in place of the usual mild steel.

Chromium Steel.—The serious character of the pitting which this steel suffers when in contact with the sea, takes away any practical interest for its use where any structures in contact with the sea are concerned. The figures applying to such conditions have been tabulated (Table XXIII and XXIV, Appendix IV) only for their interest in comparing the effects of adding different elements to steel.

In view of the beneficial effects on the pitting of ordinary iron and steel produced by previously removing the scale, the question arises as to what extent such preparation would remove this serious disability in the case of chromium steel. As the result which was presented in Table IX of the Ninth Interim Report of the Committee, and also in Dr. J. Newton Friend's report on Polished Chromium-Steel Bars in the Tenth Report, shows, its effect is in this case actually to intensify the pitting and localized corrosion.

The explanation of these peculiarities in the behaviour of chromium steel is not very apparent in the light of existing knowledge. It would seem, however, that the effect, as regards the promotion of pitting, of anodic conditions set up by differential aeration (provided this theory is assumed correct) must with chromium steel be much more marked than even the electrolytic effect between the steel and its rolling-mill scale.

The general corrosion of chromium steel, like that of ordinary iron and steel, is accentuated by removing the scale where there is actual contact with the sea; this is particularly noticeable for conditions of total immersion. Under these conditions, the loss sustained by scaled chromium steel approximates to that of the irons and ordinary steels. In air, however, previous scaling has a beneficial influence on the already low loss sustained by the material exposed with its rolling-mill scale left untouched. Pitting under these aerial conditions is negligible, whether the steel is scaled or untouched.

As regards aerial corrosion, this steel is even better than cast iron, and it shows equal freedom from pitting; compared with ordinary steel, it reduces the wastage to about one-fifteenth, a figure which is only improved upon by the 36-per-cent. nickel steel. This steel might be considered satisfactory for use under aerial conditions, although possibly with a lower carbon-content to render it more suitable for structural work. It responds to extremes of aerial temperature in much the same manner as ordinary steel, and under conditions of total immersion it is fairly constant in its wastage at all locations, and gave an especially low figure at Plymouth.

As a general observation, it may be said that the unreliability of periodical visual inspections, as a means of judging the progress of corrosion, was clearly brought out in the Committee's investigation. Such examinations have formed part of the regular procedure, but it was not until the materials were cleaned from their corrosion products at the termination of their period of exposure that a true idea of their respective merits was disclosed, this in many cases being quite different from what had been gathered during the progress of the tests. This was strikingly the case with the specimens

of chromium steel, the rusty appearance of which during exposure at Auckland was quite unprepossessing, and gave no idea of the very small wastage of this steel under aerial conditions.

It should be noted that the above remarks apply particularly to the steel with a high percentage (from 12 to 14 per cent.) of chromium. It would appear that the use of chromium in comparatively small percentages may be advantageous, especially where it is associated also with a small addition of copper. Through the kindness of Mr. Charles Mitchell, Chairman of Messrs. Dorman Long & Company, Ltd., the Authors are able to give some information regarding a steel of this type, which the firm in question manufactures. Its analysis is as follows, not exceeding in each case :—

Carbon	0·30	per cent.
Silicon	0·20	,,
Sulphur	0·05	,,
Phosphorus	0·05	,,
Manganese	0·70 to 1·0	,,
Chromium	0·70 to 1·1	,,
Copper	0·25 to 0·50	,,

This steel, it is stated, was developed more especially to meet the modern requirements for a high-tensile structural steel for marine and other purposes. It possesses a tenacity of from 37 to 43 tons per square inch and a yield-point of 23 tons per square inch, which is about 50 per cent. higher than that of mild steel. At the same time it is claimed that the steel is satisfactory as regards its workability, and is suitable for electric welding.

Its resistance to corrosion, as shown both by laboratory experiments and by practical exposure-tests, is superior to that of mild steel; *Fig. 8* (p. 46) shows the results of a comparative test on the chromium-copper steel and mild steel of British standard quality in the form of plates ¼ inch in thickness in their "as rolled" condition; namely, not heat-treated. The specimens were exposed at mean tide-level in the river Tees at Newport, Middlesbrough. At this location, the river contains about 50 per cent. of sea-water and, in addition, it is much contaminated by effluents from the various works on the riverside. At intervals of 3 months the specimens were dried, scraped free from rust and weighed.

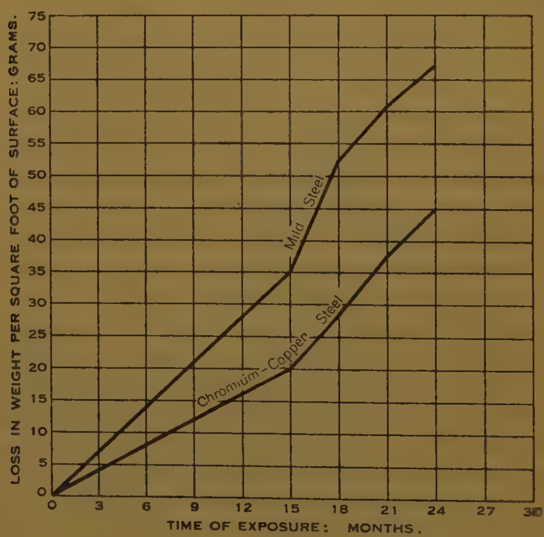
The appearance of specimens of the same materials, after exposure under the same conditions for 1 month, is shown in *Figs. 9* (facing p. 19). These were ⅛ inch in thickness, had been annealed to obviate any accidental variations due to differences in rolling, and had been pickled to remove the scale. Before the photographs were taken, the rust was removed by anode-treatment in a cyanide bath, and

the losses in weight of four specimens, two of each material, were as follows:—

Material.	Loss per square foot: grams.	
	First specimen.	Second specimen.
Chromium-copper steel	7·213	6·975
Mild steel	12·498	12·873

This represents an improvement for the chromium-copper steel of 44 per cent. over the mild steel. The lesser attack on the former steel is clearly seen in the photographs.

Fig. 8.



Relative Loss in Weight of Plates of Chromium-Copper Steel and Mild Steel immersed in the River Tees at Mean Tide-Level (Alternate Wet and Dry Conditions).

In the laboratory, and tested under alternate wet and dry conditions in a solution of magnesium chloride for a period of 3 weeks, the loss by corrosion of the chromium-copper steel proved to be 49 per cent. less than that of the mild steel.

Conclusions.—In this analysis of the results obtained from the

Committee's tests, the Authors have been particularly struck by the remarkable degree of concordance in the indications given by the various specimens of similar materials which were submitted to the different conditions of exposure. Thus, the special care taken in the planning and execution of the research have been justified in the results. It is true that it has not always proved possible to reconcile the apparently inconsistent behaviour of a particular steel in varying circumstances, but under such a variety of conditions complete consistency was not to be expected. Quite often, apparently anomalous indications are confirmed by the similar behaviour of other materials under the same conditions, which thus points to some special factor influencing the conditions. It would appear, therefore, that the explanation for peculiarities of behaviour is to be found in a more detailed understanding of the corrosive factors at work in particular cases, rather than in any irregularity of the material or of the conditions.

It should be noted how, in contact with sea-water, any special claims which a particular material may have, as regards its resistance to corrosion under aerial conditions, are greatly diminished. This levelling influence on the relative merits of two materials, or classes of material, is greater when the contact with the sea is continuous than when it is intermittent, as under half-tide conditions; this often applies also to the pitting as well as to the general wastage. Thus the possibilities of effecting improvements, by alloying or otherwise, in the corrodibility of ferrous structural materials, would appear to be much more restricted in the case of marine structures of which portions are subject to immersion in the sea, than with, for example, inland bridges or the frames of buildings. Apparently the corrosive action of salt water is such as to render the constituents, structure and physical conditions of ferrous materials much less important than they are under atmospheric conditions.

The Authors trust that the information obtained from the present analysis will be found useful in supplementing that contained in the Committee's reports, and derived from the examination made by Dr. Friend. The two methods of examination are rather different, but in so far as the ground covered is common to both, the conclusions arrived at appear to be in excellent agreement.

ADDENDUM.

After the present Paper had been prepared, and with the publication by the Sea-Action Committee of their fourteenth Report, completing the results of the 10-year period of exposure on duplicate specimens, it has been possible to make an analysis of these figures

similar to that recorded earlier in the present Paper for the 5-year tests. A study of the Tables which have been prepared from the 10-year results, and which correspond to Tables IX to XXIV (Appendixes III and IV), leads to practically the same conclusions, which therefore receive excellent confirmation.

It is interesting to find that, in general, the maximum depth of pitting of the ordinary steels and irons has only very slightly increased in this further period. The depth of pitting thus tends to reach a limit, which is appreciably less when the material has been previously scaled.

A further feature of note is that the 10-year specimens do not show the same excessive pitting under half-tide conditions at Halifax that was observed with the 5-year specimens. In view of the excellent agreement which exists in the results for the 10-year and 5-year series of tests in most other respects, it would appear that this alteration in behaviour may be accounted for by some favourable change having occurred in the conditions operating at Halifax since the earlier period of the exposure. As the general corrosion under half-tide conditions at this location is also far from having doubled in the 10-year period, and on the other hand the progress of corrosion under aerial and half-tide conditions has been maintained more or less consistently with the other locations, the circumstances point to the change being one which more particularly affects the surface condition of the sea-water.

The relative merits of the different materials, as based on the 10-year examination, remain much the same as before, although the comparative figures undergo some modification. The rolled irons as a class have practically lost the small advantage which they had over the ordinary steels in general wastage, but the Low Moor iron "N" remains distinctly the best of these two groups. Of the cast irons, the hot-blast iron now shows slightly the better figures on the average, although the difference between it and the cold-blast iron is only slight after each period of exposure. Except for aerial conditions, these materials would also be judged rather from their propensities to graphitization, which do not seem to differ greatly, than from figures of wastage.

Practical Exposure Trials.

Exposure Tests on Chromium Steel in River Water.

In August, 1923, it became necessary for those responsible for the building of the Sennar Dam on the Upper Nile to consider the nature of the materials to be used in its construction, and the desirability of making use of the non-rusting character of chromium

steel came into consideration. Although the complete substitution of chromium steel for mild steel could not be contemplated on account of the cost, it was felt that steel of the former type might, subject to satisfactory evidence of its suitability, be employed for certain portions, such as the stanching bars of the sluices. It was, therefore, decided to make a preliminary exposure test under conditions as similar as possible to those which would operate at the dam; the record of these tests, which were carried out by Messrs. Ransomes and Rapier in collaboration with the Authors, is shown later.

Materials Tested.—The specimens tested comprised, under the distinguishing letters "A" and "B" respectively, representatives of both the hard and soft grades of chromium steel which are included in the Committee's research. In addition, a further material, denoted by "C," of the soft type, but containing a relatively high proportion of silicon (2·4 per cent.), was tested. Full particulars of the materials are given in Appendix V, Table XXV, including their analysis, heat treatment, and the condition in which their surfaces were exposed in the different specimens, which were in the form of bars 3 inches by ⅜ inch in section, and 12 inches long.

Methods of Exposure.—With the exceptions noted later, two separate tests were applied to each specimen, and for this purpose they were cut into two portions.

A further type of test, described under section (2*a*) on p. 50, was also applied to one of the specimens of the soft-grade material. Details of the respective methods are as follows:—

(1) *Exposure in vessel containing river water, which was allowed to evaporate.*—(*a*) As a simple and preliminary test, a 3-inch length was sawn from each bar, with the exception of "A.1," "A.4," and "B.3," and a portion of one of the flat faces was roughly milled. This piece of bar was placed vertically in a jar containing water from the tidal stream of the river Orwell, the milled portions being uppermost; the river water at this location, besides being saline, is contaminated by sewage, the salinity amounting to 1,740 parts of chlorine in 100,000 parts.

The water was allowed to evaporate and was renewed every two months, and thus the specimens in each such period became totally exposed to the air during a part of the time, the upper milled portions being exposed the longest. The actual exposure took place at several intervals in the period between September, 1923, and July, 1925, and occupied, in all, 20 months.

(*b*) The remaining portions of the cut bars, and the whole of "A.1"—which, owing to its hardness could not conveniently be cut up—were also subjected to exposure by immersion in the same tidal

water of the river Orwell in the following way. A rack of teak-wood was constructed, which supported the specimens vertically at the bottom and along one edge, and the wooden frame along these edges covered the test material for a width of about $\frac{5}{8}$ inch. The frame, with the specimens, was placed in a drum containing the river water, with the specimens totally immersed, and when all the water had evaporated, a fresh supply was poured in, renewal being made in this way as often as necessary during the progress of the test, which lasted 553 days. Specimens " B.4 " and " B.5," however, were only exposed for 535 days.

On removing all the specimens on 14 March, 1924, all loose scale and corrosion-products were removed to facilitate an examination. As the result, it was considered desirable to extend the period, and further exposure was made, up to the total recorded above.

(2) *Tests of chromium steel* (*soft grade*) *in contact or metallic connection with other metals.* (*a*) Since it was known that chromium steel is to some extent protected against corrosion by contact or metallic connection with some other metals, tests were carried out in the following manner to determine the extent of this protective action, the other metal being mild steel or cast iron.

For the first set of tests, carried out at Ipswich, the chromium-steel specimen selected was " B.3," that is, one of the annealed bars which was prepared with a pickled surface. Representative specimens of ordinary mild steel and cast iron were also provided.

The specimens, in the form of small pieces cut from the larger bars, were totally immersed in river water in loose contact in pairs as shown in Appendix V, Table XXVI ; single detached specimens of the three materials were also immersed. After 5 days' immersion, the pairs of specimens were joined up with copper wire to ensure better metallic connection, and the test was then allowed to proceed over a period of about 7 months.

(*b*) In a similar set of tests carried out at the Hecla Works, Sheffield, the composition and other particulars of the material were as shown in Appendix V, Table XXVII.

The specimens, $2\frac{3}{8}$ inches by 1 inch by 0·04 inch thick, were held vertically, with their lower ends 1 inch in the corroding liquid, while the two members of each pair were spaced 1 inch apart and joined by a copper wire outside the liquid. Separate tests were carried out both in artificial sea-water (a 3 per cent. solution of Tidman's sea-salts), and in Sheffield tap-water, the exposure occupying 14 days in each case.

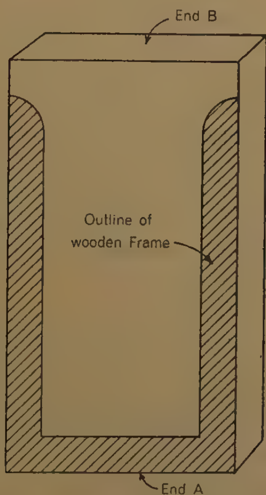
Results of Tests.—Dealing with the separate tests in the order as described above :—

1. *Exposure in vessel containing river water, which was allowed to*

evaporate.—(*a*) The condition of the specimens after exposure in the first series of tests is shown in Appendix V, Table XXVIII.

Except in the case of specimen "B.1," which was tested in the "as rolled" condition, and which gave results distinctly below the average, none of the materials tried, nor any heat-treatment or initial condition of surface, has given results outstanding from those of the others. It would, therefore, appear that, for practical purposes, advantage could be taken of the comparative softness and machinability of the softer grade of material in its annealed condition.

Fig. 10.



Chromium-Steel Specimens after Exposure to River-Water, Showing Outline of Teak Frame (Appendix V, Table XXIX).

No benefit seems to have been derived from an additional percentage of silicon, as represented by material "C."

(*b*) For the second series of tests, in which the test-pieces were held in a teak frame, the condition of the specimens after exposure is shown in Appendix V, Table XXIX. One special feature observed is the very pronounced corrosive attack at the contact of the specimens with the wooden supports, leaving in many cases a clear outline of the support, as shown in *Fig. 10.*

Pitting is general with all the specimens, whether of the "hard" or "soft" type, and none of them has proved entirely immune from attack. For the majority of the specimens there is not much to choose in their behaviour, except that it is clearly detrimental to retain the original rolling-mill or heat-treatment scale

on the material. The omission of any heat-treatment, that is, exposing the material in the "as rolled" condition, results in increased corrosion in the case of the softer-grade specimen "B.1." The "as forged" specimen "A.1" of the harder grade has, on the other hand, behaved fairly well.

Of all the specimens the one which has behaved the best is "A.4," representing the harder material in its quenched and tempered condition, with its surface polished. The improvement in the corrosion-resisting properties of this material when prepared in such a way is well known, and is well borne out by the results of these tests; it should be noted, however, that even in this case there is the same characteristic tendency to pitting.

As in the previous series of tests, material "C," with 2·4 per cent. silicon, seems to offer no advantage over "B," which has only 0·5 per cent. of this element.

2. *Tests of chromium steel* (*soft grade*) *in contact or metallic connection with cast iron and mild steel.*—(*a*) The results of these tests are shown in Appendix V, Table XXVI. When tested separately, the cast iron suffered rather more corrosion, as measured by loss of weight, than the mild steel, while the chromium steel was superior to either in the ratio of about 5 to 1.

The figure representing the losses incurred by exposing the chromium steel in contact with either mild steel or cast iron is very small in both cases. On the other hand, the corrosion of the cast iron and mild steel was increased by roughly 50 per cent. Thus the protective value of mild steel on chromium steel, as seen in the Sea-Action Committee's Experiments, is confirmed under the present conditions, and cast iron has a similar effect. It is interesting to note that the total corrosion is appreciably increased in the case of either pair of materials when contact is established between them; that is, the reduced loss of the chromium steel is more than counterbalanced by the increased loss on the cast iron or mild steel. The contact between cast iron and mild steel, however, resulted in reduced total corrosion, each of the materials apparently receiving some degree of protection from their contact with the other.

The tests also included a triangular connection between the three materials, and in this case the chromium steel was protected to the same extent as before. The cast iron and mild steel, however, show very much less corrosion than when each was connected separately with the chromium steel. Their losses under these triangular conditions are, in fact, approximately similar to those obtaining when they are exposed separately.

(*b*) The results of the tests of a similar character which were carried out at the Hecla works are shown in Appendix V, Table XXX. The

corroding media, namely, tap-water and artificial sea-water, are somewhat different from that in the tests just recorded, which were carried out in water from the river Orwell, and the exposure was over a much shorter period. The greater corrosive action of sea-water, as compared with tap-water, is readily seen from the figures, especially in the case of the chromium steel.

Here, again, the protective action of the cast iron and mild steel upon the chromium steel is very marked in both media. The corrosion of the mild steel is increased by such contact in both cases, and the combined amount of corrosion of the two materials is also increased to some extent. In the case of the cast iron, however, its corrosion is not increased by contact with the chromium steel, and under exposure to tap-water it is definitely decreased. These tests, both in tap-water and sea-water, show a decrease in the combined amount of corrosion. As in the tests carried out at Ipswich, both the mild steel and cast iron are to some extent protected by contact with each other, and this is particularly the case with the mild steel in sea-water.

On the whole, the indications of the separate sets of tests, bearing in mind that they were with entirely different specimens and in different media, may be considered as being fairly consistent.

Behaviour of Chromium Steel in the Sluices of the Sennar Dam.— As a result of these investigations, the suitability of the softer grade of chromium steel (" rustless iron ") for the stanching bars of the sluices of the Sennar Dam was considered as reasonably established, and about 4 tons of the material was ordered for this purpose. Ten sluices were so fitted, Nos. 41 to 50 inclusive.

The type selected was that designated in the present research as " B," specified to be in the annealed condition with surface pickled, corresponding with the specimen " B.2 " above and with the Sea-Action Committee's designation " U." A tensile test made on the material, as supplied, gave the following figures :—

Yield-point	23·5 tons per square inch
Maximum stress	37·0 ,, ,,
Elongation	34·5 per cent.
Reduction of area	62·6 ,,

Through the kindness of the Chairman of the Sea-Action Committee, Mr. M. F.-G. Wilson, M. Inst. C.E., whose firm were the Consulting Engineers to the Egyptian Government for the construction of the dam and sluices, the Authors are able to give the report of the behaviour of the steel in this service, as obtained from Mr. R. M. Macgregor, of the Irrigation Advisor's Office of the Sudan

Government. The Report, dated the 22nd June, 1931, states that, owing to the work on the aprons being in progress, these gates could not be lifted in 1929 and 1930. Allowing time for the last of the season's concrete to set, one of these gates, No. 43, was lifted in June, 1931, for the first time for 3 years, and compared with No. 40 fitted with the ordinary steel bar. The following quotation is from the report :—

"The rustless steel was in places slightly discoloured with rust, but it seemed as if this might have arisen from contact with the cast iron groove. There was no sign of corrosion, and the surface for the most part showed the metallic lustre and general condition with which it left the planing machine. The plain steel on the other hand was heavily corroded and rusted. We were all struck by the difference in condition of the two steel surfaces. It may be taken quite definitely that the rustless steel is in fact substantially rustless."

Thus, the indications of the preliminary exposure trials appear, in this case, to have been borne out in a very satisfactory way, and the choice of the material, based on these trials, has been fully justified. It is, however, especially interesting to note that, when subjected to the action of sea-water under conditions of total immersion in the Committee's Research, this same steel under the designation "U" behaves in anything but a satisfactory manner, a $\frac{3}{8}$-inch bar being completely perforated by corrosion within a year.

Exposure Tests on Marine Buoys.

In 1922, by kind permission of the Elder Brethren of Trinity House, a series of specimens was attached to certain of their buoys in selected localities. The research was not, in this case, directly connected with any specific difficulty experienced with the material for buoys or other structures and appliances under the control of Trinity House. With the careful attention which is devoted to the buoys by periodical inspections, repainting and varnishing, it is understood that no serious troubles due to corrosion are experienced. Naturally, however, like other marine authorities, the Elder Brethren are interested in any efforts to deal with the corrosion-problem, and readily gave permission for the tests to be carried out in this way as a convenient means of obtaining useful information.

In consultation with the Engineer-in-Chief, Mr. D. W. Hood, the buoys selected for the tests were three in number :—

The "North Brake" buoy off Ramsgate, representing open sea conditions.

The "Nore Sand" buoy in the sea-reach of the Thames estuary, providing conditions representative of those existing in estuaries where the water is rather less saline, and may therefore be expected to be different in its corrosive effects.

The "North Cliffe" buoy in the harbour at Harwich, where, naturally, rough-weather conditions are not so severe as at the other sites. Consequently, by comparison with those on the unprotected "North Brake" buoy, the specimens should provide some evidence of the special effect, if any, of stormy conditions on the corrosion of steel exposed in the region of the water-level.

Some details of the specimens used for the research, and their method of exposure, were contained in Section VI of a paper [1] by Sir Robert Hadfield. At that time the results were not available and the full particulars are now published for the first time.

Form of Specimens and their Method of Attachment to the Buoys.— The type of buoy in each case was the same, and is described as a steel drum buoy. By adopting a bent shape, the specimens were exposed all round their four sides for the greater part of their length, and could be attached by riveting. At the same time, the flat section of the specimens provided a large surface in relation to their weight. To minimize electrolytic action, a washer of hard rubber was inserted between each end of the specimens and the buoy before they were secured with mild-steel rivets. No paint or other coating was applied. The placing of the specimens was similar on each buoy; they were in sets of four at two different levels, one above and the other below the still-water level. Those in the set under water were duplicates of those above, and similar sets were attached to each of the buoys. They were each marked with a distinctive mark by stamping on their ends, and in view of the possibility of these becoming obliterated by corrosion, special note was taken of the relative positions of the pieces.

By the action of the waves and spray, and when rain was falling, the upper specimens were sometimes wetted, and even immersed in a spasmodic manner, but there would be long intervals in which they would become dry and remain in this condition, while in rough weather, those in the lower set were at times momentarily out of the water. The conditions, therefore, were in both cases representative of those experienced by much of the steel exposed to sea-action.

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1 "The Corrosion of Ferrous Materials, with Special Reference to their Resistance to the Action of Sea-Water," XIIIth International Congress of Navigation, London, 1923.

The horizontal position of the specimens ensured more uniform conditions for each, especially in the upper sets; with a vertical arrangement their lower ends would have received more frequent and longer immersion than the upper ends, and would have received and accumulated some of the water draining off the upper parts.

Materials Tested.—The test was confined to a determination of the relative merits of chromium steel of the harder grade, included in the Committee's research under the designation "J," in comparison with ordinary mild steel. As will be seen from the particulars given in Appendix VI, Table XXXI, the specimens of chromium steel were so prepared as to provide information regarding the effect both of the condition of its surface and of its heat-treatment on the corrodibility of this material.

The heat-treatments applied were :—

(1) A simple softening treatment by cooling from 650° C. in air.

(2) Annealing by heating to 775° C. and cooling in the furnace.

While a quenching and tempering treatment usually gives the best results for this steel, such treatment would generally be impracticable in the forms required for structural purposes. Further, as the number of specimens possible was limited, it was considered that annealed specimens would provide more useful information.

Results of Tests.—The buoys were fitted with the specimens, taken to their respective locations, and put into service. By arrangement with the Engineer-in-Chief, special notes were taken of the visual condition of each of the specimens during the routine inspection of the buoys, and the continuity of the conditions of exposure was left as far as possible undisturbed by these examinations; that is, the specimens were not cleaned. The dates of the commencement of exposure and of the examinations for each set of specimens, as well as the notes taken at the visual examinations, are shown in Appendix VI, Table XXXII.

The period of duration of the tests was determined by the failure of some of the rivets due to excessively rapid corrosion, brought about by their contact with the non-rusting steel. Evidence of such action is provided by the tests in the previous part of this Paper.

At the examination in December, 1923, of the "North Brake" buoy, it was reported that the two rivets at the end of one of the underwater specimens ("G.6") were missing, and a few days later, the buoy having been found waterlogged and in danger of sinking, it was replaced by another and was brought into harbour.

In February, 1924, it was further reported that the rivets holding

the specimens on the "Nore Sand" buoy were in a badly corroded condition, and that some of those in specimen "G.16" had disappeared; in these circumstances this buoy also was withdrawn and replaced, and in June, 1924, a similar occurrence happened with the remaining buoy.

Without an entire rearrangement of the method of attachment, which was found to be impracticable, this very rapid corrosion of the rivets could not be avoided, and it was therefore necessary to discontinue the tests. The specimens were each subsequently cleaned with a hard brush and soap to remove loose corrosion products, and their loss of weight during exposure was determined. The losses are recorded in Appendix VI, Table XXXII.

Corrosion of mild steel at the different locations.—It is interesting to see that, so far as the above-water specimens are concerned, and having regard to the respective periods of exposure, the mild steel has corroded to much the same extent whether under open-sea conditions, under the more protected conditions in the Thames estuary, or even in harbour.

The underwater specimens, on the other hand, are distinctly more corroded at the locations where the water is more fully saline (namely, in the open sea and in the harbour at Harwich), than in the estuarine water of the Thames.

The indications of these tests are therefore that the worst conditions for mild steel, among those to which it has been subjected, are with complete immersion and where the water is of full salinity; the most favourable conditions are with complete immersion in estuarine waters.

Behaviour of the chromium steel.—The strikingly superior resistance of the chromium steel as compared with that of mild steel, which was observed in the visual examination, is fully confirmed in the figures for loss of weight. This applies under all the different conditions of exposure, and for each of the different physical conditions in which the chromium steel was tested.

Owing to the smallness of the figures for the chromium steel, and the relative importance therefore of the experimental errors, any evaluation of the respective merits of these different physical conditions is problematical. The figures in the sixth and seventh columns of Table XXXII, Appendix VI, however, show that a somewhat greater loss has occurred with the specimens exposed with their scale untouched than in the case of those which have been cleaned. Whether this is actually due to the attack being greater on account of the presence of scale, or otherwise, is not clear; probably, however, such attack as there has been on the scaled specimens was, in the comparatively short period of the tests, only sufficient to loosen the

scale, and the main portion of the loss in this case is represented by the scale so removed.

A further feature of the results, both in the appearance of the specimens, and generally from the figures for the loss in weight, is the slightly greater corrosion of the chromium-steel specimens in the above-water position, as compared with those under water; this is sufficient to reverse the order of the results obtained with the mild steel.

Exposure Tests in an Urban Atmosphere at Birmingham.

In May, 1921, the Authors decided that it would be useful to have some indication of the relative behaviour, under the more ordinary conditions existing in the atmosphere of a large city, of the various ferrous materials tested by the Sea-Action Committee, as this Committee itself was concerned more particularly with marine corrosion. For this purpose, portions of bars were utilized which were insufficient in length to provide the full 2 feet necessary to make the standard specimens adopted by the Committee.

Materials Tested.—The materials selected are shown in Appendix VII, Table XXXIII, and comprise all the plain carbon steels as well as the alloy steels; all the specimens were allowed to retain their rolling-mill scale as in the case of the Committee's research. Further particulars of these steels are given in the Paper [1] describing their preparation.

Method of Exposure.—Through the kind co-operation of Dr. J. Newton Friend, the specimens were exposed on the roof of the Technical College, Birmingham. The bars, ten in number, 12 inches in length, and previously weighed, were supported vertically in a wooden frame, into which their ends were recessed loosely for a depth of 1 inch, and the frame was then placed so that the bars were completely unprotected on all sides from both wind and rain.

The specimens were exposed continuously for 6 years from May, 1921, to May, 1927. On their removal the loose scale was taken off and the loss of weight determined, precisely the same procedure being adopted by Dr. Friend in this case as for the specimens in the Committee's research.

Results and Conclusions.—The results, together with Dr. Friend's description of the appearance of the specimens after cleaning, are given in Appendix VII, Table XXXIII. For comparison with the Committee's results for the marine corrosion of these materials,

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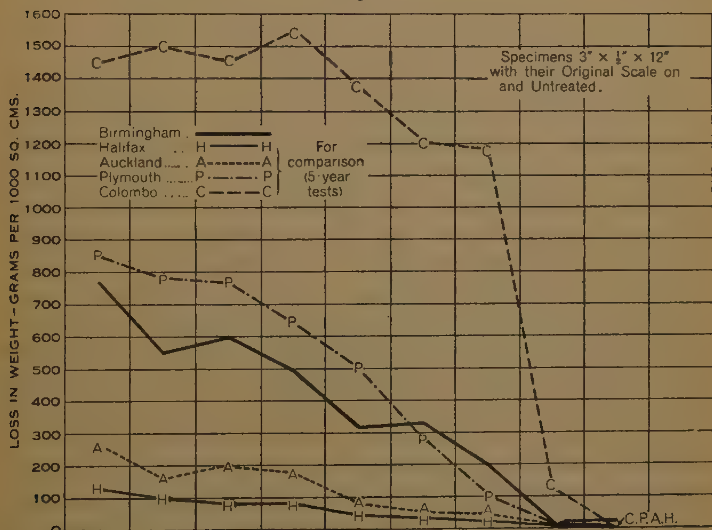
[1] Footnote on p. 4.

the figures for the loss of weight have been worked out on the same basis, namely "grams per 1,000 square centimetres." The area exposed in the present instance is taken as that included in the length of 9¾ inches between the supports. Some corrosion actually occurred outside these limits, specimens "B" and "D" in particular being definitely attacked at the ends which were in the wooden

Fig. 11.



	B	F	D	E	G	H	K	J	L	
	Mild Steel Low Mn High S & P	Mild Steel with 0·7% Mn	0·40% C Steel	Medium Carbon Steel Low S & P	Mild Steel + 0·5% Cu	Mild Steel + 2% Cu	3½% Ni Steel	13% Cr Steel	36% Ni Steel	
C	0·21	0·24	0·40	0·33	0·21	0·22	0·31	0·36	0·09	C
Si	0·17	0·16	0·20	0·20	0·14	0·14	0·18	0·22	0·09	Si
S	0·090	0·058	0·049	0·021	0·043	0·033	0·029	—	—	S
P	0·060	0·048	0·045	0·027	0·046	0·041	0·038	—	—	P
Mn	0·30	0·68	0·85	0·80	0·94	0·90	0·54	0·13	0·77	Mn
Cr	—	—	—	—	—	—	—	13·57	—	Cr
Ni	—	—	—	—	—	—	3·75	—	36·56	Ni
Cu	—	—	—	—	0·63	2·18	—	—	—	Cu

Corrosion Tests in the Atmosphere at Birmingham on Steels Selected from the Research of the Sea-Action Committee of The Institution.

recesses. Apart from the difference in the period of exposure, the figures are, therefore, slightly higher relatively than they should be for strict comparison with the Committee's results.

In Table XXXIII, Appendix VII, and *Fig. 11*, the Committee's materials are shown in increasing order of merit, according to their average figures for loss in weight under aerial conditions at the various locations in the Committee's research. Compared with the results

from the Committee's research, it will be seen that the relative behaviour of the materials, under conditions of exposure in the urban atmosphere at Birmingham, is substantially the same as in marine atmospheres. The difference in the relative behaviour of the materials as found at Birmingham and at Plymouth is not, in fact, any more pronounced than that between Plymouth and Auckland or Halifax (Nova Scotia). At Colombo, however, where corrosion occurs to an excessive extent, there are several differences to be noted in comparing the relative merits of the various materials, and also in the ratio of the losses where the order is the same.

A feature, however, of definite contrast between the behaviour at Birmingham and at the Committee's stations, including Colombo, is the fact that, of the copper steels, "H," with 2 per cent. of copper, does not at Birmingham show the same superiority over "G," which has only ½ per cent. of copper, as elsewhere, but is if anything slightly inferior. In other comparisons, the relative behaviour of any two materials, as seen at Birmingham, will be found reflected at one or more of the marine stations. The general corrosion of most of the materials at Birmingham is rather less than at Plymouth, and since the test lasted 6 years instead of 5, the conditions at Birmingham may definitely be said to be more favourable than the marine atmosphere at Plymouth; they are not, however, so favourable to the special steels as to the ordinary steels.

The trials thus help to confirm the Committee's assessment of the relative merits of the various steels, and to broaden its practical application. The type of steel represented by "B," with high sulphur- and phosphorus-contents, again proves very unsuitable where aerial corrosion is concerned, and the alloy steels are superior to the carbon steels. The chromium steel "J" again shows a very small general wastage, fully bearing out its excellent qualifications for aerial conditions such as these, where its pitting propensities are not in evidence. In these tests it has, as regards actual loss in weight, proved even better than the 36-per-cent. nickel steel "L," which also confirms the excellent behaviour it displayed in the Committee's research; and its appearance after exposure is superior to that of the chromium steel. Steel "K," with 3½ per cent. of nickel, is next in order among the Committee's steels, while the addition of only ½ per cent. of copper to the steel cuts down the wastage, in the present instance, by about 40 per cent. as compared with mild steel. In the purer atmosphere of rural districts this degree of superiority is probably not to be expected, as the Authors have already shown.

Corrosion of Specimens Totally Immersed in the Sea in the Gulf of Paria.

The necessity for laying an oil pipe-line on the sea bed of the Gulf of Paria led, owing to the especially corrosive conditions believed to exist there, to the following tests being carried out on a variety of steels, by immersing them on the site.

The severity of the corrosion experienced is illustrated by an instance reported by the United British Oilfields of Trinidad, Ltd., where a $\frac{5}{16}$-inch mild-steel plate forming the bottom of a storage tank was eaten through in 9 months, and when it was drying off after the removal of the tank it became quite hot due to the chemical action still going on. During the flood-periods of the river Orinoco, the sea-water in the gulf is contaminated by large quantities of organic matter, chiefly vegetable, although in the intervals it is comparatively fresh. Near Point Fortin there are outcrops on the sea-bed of pyrites-bearing clay, in the direct line of the route of the proposed pipe, and it was at this location that the mild-steel tank, referred to above, was immersed. The present tests on specimens prepared by the Authors were carried out by the United British Oilfields of Trinidad, Ltd.; high-chromium (13 per cent.) steel had already been tested and found unsatisfactory. Two series of tests have so far been made, the second intended to amplify or confirm information obtained in the first.

First Series.

Materials Tested.—As a standard of comparison, a basic open-hearth mild steel, designated No. 3042, was chosen; this is of the type ordinarily employed for the manufacture of pipes. The Sea-Action Committee's research had clearly shown that, for conditions such as those of total immersion in the sea, alloy steels, while in general showing a superiority over plain carbon steels, are not so effective in reducing wastage as they are under half-tide or aerial conditions. No very striking improvement could therefore be expected in the present instance, unless the specially corrosive conditions enabled the alloy steels to show a greater superiority than in more normal circumstances.

The various alloy steels tested, and their analyses, are included, with others tested in a later series, in Appendix VIII, Table XXXIV; each of these has special claims for its resistance to corrosion. The copper steel, No. 3714, and the 36-per-cent. nickel steel, No. 3450, are of the same types as the materials known as "G" and "L" which have already proved their merit in the Committee's research, while No. 3754 is of the well-known "18–8" composition, now widely used for many purposes where non-rusting steel is required; in all there were six specimens in this series.

Method of Exposure.—The specimens were in the form of small plates from $\frac{1}{8}$ inch to $\frac{3}{16}$ inch in thickness, and varied from 5 inches by 4 inches to 6 inches by 4 inches in superficial dimensions; their identity was preserved by a small hole or holes drilled through in various positions.

After a record had been taken of their weights, the six specimens were sent to Trinidad, where they were mounted in a block of hard wood; they were laid flat in a row in the block, with $1\frac{1}{4}$-inch spacing, each plate fitting into a recess so that its surface was flush with the surface of the wood. Two thin wooden battens running the length of the block secured the edges of the specimens; the sea-water had, therefore, free access to the front of the specimens, but the back surface was only reached by the water which percolated through the interstices between the specimens and the block.

The block, complete with the specimens, was lowered into the sea by means of an oil-well sucker-rod about $1\frac{3}{4}$ miles from the shore in the Gulf of Paria, at a mid-tide depth of 26 feet 11 inches. The block was upright, with specimen No. 3450 in the uppermost position, the lower specimen being about 5 inches from the sea bed, and the uppermost 36 inches from it.

The specimens remained undisturbed for 63 days, from 11 June to 13 August, 1928, and when they were removed they were already encrusted with barnacles, coral formations, and sea-weed, with numerous small crabs and other organisms. There was an appreciable quantity of mud around the edges of the plates and between them and the wooden battens. The barnacles and other adhesions were cleaned from the plates by scraping them with a dull knife, and a visual examination was then made of the exposed surfaces.

The block was returned to the sea on 15 September, in the same location but with the specimens in the reverse order, namely, with No. 3450 in the lowest position, to counteract as far as possible any favourable or unfavourable effects due to relative position. On 1 December, after a further period of 76 days (making a total of 139 days (about $4\frac{1}{2}$ months)), the specimens were again removed. The incrustation on the front surfaces was even heavier than that after the previous exposure, and in addition to the molluscs, there was a fair amount of rust, varying in quantity and colour with the different specimens from red to buff, while in some cases small black patches of the original scale remained. The back surfaces of the specimens appeared to be comparatively unaffected, and on specimens Nos. 3450 and 3754, and on the mild steel the patches of scale were fairly large. The edges protected by the battens were also fairly clear of corrosion products, but in the case of the steels

Nos. 3754, 4344 and 4375, which contained chromium, distinct pitting was already visible on these edges before cleaning. The wood had been attacked in several places by *teredo*, and under specimens Nos. 4344 and 4375 there were tunnels about 3 inches long.

The specimens were detached from the block, and after their surfaces had been cleaned in the same manner as before, they were returned to the Hecla works, Sheffield. After a visual examination, they were cleaned with hard soap and water to remove the loose products of corrosion, and were again visually examined and weighed.

Results.—The results obtained are all included in Appendix VIII, Table XXXV, the losses in weight per unit area being reckoned in grams per 1,000 square centimetres of exposed area, as in the Committee's research.

So far as can be judged from a rather indirect and imperfect comparison of the behaviour of the mild steel in both this and the Committee's tests, the reputed severity of the conditions in the Gulf of Paria seems to be borne out. At the various harbour-locations, Halifax (Nova Scotia), Auckland, Plymouth, and Colombo, the loss in 5 years for the various grades of mild steel varied between 325 and 459 grams per 1,000 square centimetres, or a rate of from 65 to 92 grams per 1,000 square centimetres per year. In the present test, although the mild-steel specimen was not exposed to the full action of the sea on all sides, and the corrosion products could not be completely removed, it showed a loss of 58·9 grams per 1,000 square centimetres in 139 days. At the same rate this would amount to 154 grams per 1,000 square centimetres per year, or about double that observed in the Committee's research. However, taking the results as obtained in this trial, it is interesting to see how the various special steels have behaved.

The copper-steel specimen, No. 3714, with rather less copper (namely, 0·39 per cent.) than material " G " (which had 0·63 per cent.) in the Committee's research, shows a greater loss than ordinary mild steel; the attack is, however, more uniform, and if allowance is made for the probable under-estimation of the loss on the mild steel, the copper steel is possibly somewhat superior as regards wastage.

The 36-per-cent. nickel steel No. 3450 is undoubtedly the best of the materials tested, its rate of wastage, which is about one-half that of mild steel, confirming the experience obtained in the Committee's research in this respect, while it exhibits the same characteristics of uniform attack, with comparative freedom from pitting.

Although the 29-per-cent. chromium steel, No. 4344, and steels Nos. 3754 and 4375 with high contents of both nickel and chromium, also show favourable results as regards their total wastage (the " 18–8 " type, No. 3754, being in fact superior in this respect even

to the nickel steel), they are badly pitted by local corrosion. This is a dangerous factor, which would probably put a short period to their service-life, particularly in the case of pipe-lines.

Second Series.

Materials Tested.—In May, 1931, to confirm and extend the results of the first series of specimens, a second and larger set of fifteen specimens, as shown in Appendix VIII, Table XXXVI, was sent for exposure.

Although the alloys with high chromium-contents had shown bad pitting characteristics, it was thought useful, because of their small general wastage, to try a few more representatives of this class, which appear in the table under the numbers 3765, 4310 and 3760. In this second set further specimens of the 36-per-cent. nickel steel No. 3450, which had behaved so well in the first series, were also included, with a steel having the comparatively low nickel percentage of 3·75, corresponding to " K " in the Committee's research.

A specimen of the same mild steel, No. 3042, was used as the standard of comparison, as in the first series of trials. Further, in view of the interesting results obtained from a series of steels containing small percentages of both nickel and copper,[1] it was also decided to include representatives of this type of steel in the tests. The specimens of this alloy steel were numbered 3236, 1251A, and 1251B.

Method of Exposure.—The same general procedure was adopted as with the specimens in the first series, and they were exposed for one continuous period of 6 months. With the molluscs still adhering to one face, they were returned to the Hecla works, where a visual examination was made and the specimens were afterwards cleaned, freed as far as possible from any loose corrosion products, and weighed. A few weeks after the cleaning, the specimens in the interim having been left exposed to the air in the laboratory, further portions of the surface were found to be flaking off from some of them; they were therefore all rubbed with a wire brush and reweighed.

The presence of a complete covering of molluscs may perhaps be regarded as taking the problem rather out of the field of corrosion, and introducing the possibility that much of the wastage is due to some action of the organisms themselves. As the record shows, a distinct pattern was left on specimen No. 3765 after cleaning, outlining the original forms of separate barnacles. Such a pattern might naturally be formed owing to the greater freedom of access of the sea-water between the barnacles, without reference to any

[1] Dr. J. N. Friend and Mr. W. West, " Resistance of Copper-Nickel Steels to Sea Action." Journal Iron and Steel Inst., vol. cxxiii (1931, No. I), p. 501.

possible attack by secretions from the barnacles themselves. There was no evidence of such an attack on any of the specimens of the definite character recorded in the Committee's research.

Whether the problem of the wastage of steel structures underwater is regarded as being in the field of corrosion, or of biology, or of both, the presence of molluscs appears to be inevitable, and from whatever cause the wastage occurs, it must naturally be taken into account in any efforts to improve the life of steel structures under such conditions.

Results.—The results obtained from this second series are given in Appendix VIII, Table XXXVI. The behaviour of the mild-steel specimen used as a standard of comparison, which has corroded at about the same high rate as before, tends further to confirm the specially corrosive nature of the sea-water in the Gulf of Paria. A comparison of the rates of corrosion of the special steels Nos. 3450, 3611, and 3714 with those of their counterparts, "L," "K," and "G" in the Committee's research, leads to a similar conclusion.

Regarding the results from those alloy-steel specimens which were exposed, like the mild steel, with their scale untouched, the copper steel, No. 3714, again shows rather less wastage than the mild steel. In the present instance the copper steel has also shown a propensity to localized attack. The 36-per-cent. nickel steel No. 3450 has shown the same consistent and excellent behaviour as before, with a loss of only about one-half that of mild steel, and has suffered a uniform attack. The nickel steel No. 3611 containing only a small amount (3·75 per cent.) of the added element, shows, as it did (*cf.* "K") in the Committee's research, only a comparatively small advantage over mild steel in its total loss by wastage; this amounts to 23 per cent. in the present instance, and it is not so uniformly attacked.

The representatives of the general type of corrosion-resisting steels, containing high chromium with varying amounts of nickel and other additions, are Nos. 3765, 4310 and 3760 in the present series. These representatives, like the others previously tested, show quite attractive figures for their total loss by wastage, but their performance is again seriously marred by excessive pitting or local attack. Although excellent steels for many other circumstances, their use could not on this experience be recommended for underwater structures.

Steels Nos. 3236, 1251A and 1251B, containing comparatively small percentages of both nickel and copper, do not under these conditions show the advantage which might have been anticipated, in view of the beneficial effects usually observed for these elements when used separately as additions to steel. No. 1251B contains

a total of 5·8 per cent. of the two elements combined, but actually No. 3236, with only 1·71 per cent. of nickel and 0·77 per cent. of copper, appears to be the best, although it does not seem to offer a sufficiently improved performance over that of mild steel to compensate for its extra cost in actual service.

With five of the materials, including the 36-per-cent. nickel steel and the copper steel, opportunity was taken to test, by the addition of further specimens, the effect of previously removing the scale by pickling or sand-blasting. The effect of this procedure on the copper steel has certainly been, as with ordinary steel in the Sea-Action Committee's research, to make the attack more uniform. The results are not, however, available in this case to show whether the wastage is actually lessened or increased. The steels with a high chromium content, Nos. 4310 and 3760, are distinctly the worse for the previous removal of their scale; the characteristic localised attack is intensified, and the total wastage is increased, in the same manner as in the case of the plain 13-per-cent. chromium steel in the Committee's experiments.

Specimen No. 3765, on the other hand, gives a very much improved performance; the tendency to pitting is greatly minimized by scaling, and the total loss by wastage is reduced to an almost insignificant figure. It will be noted that although of the same general type as Nos. 4310 and 3760, this steel has a lower content of chromium and a considerably higher content of nickel; the latter element has been shown to have a beneficial effect by the excellent results of the 36-per-cent. nickel steel.

For those purposes where it is possible to prepare the surface before use, the results make this steel (No. 3765) look attractive. Further confirmation, however, is necessary, in view of the serious pitting which has resulted in the present experiments, and also in every other similar case of which the Authors have had experience as regards steels of high chromium-content (namely, 12 per cent. and upwards), whether these contained nickel or not. The reduction of the chromium-percentage in specimen No. 3765 to 10 per cent. hardly seems a sufficient safeguard against such serious pitting, and even in this case pitting on the edges is not entirely absent. The behaviour of the 36-per-cent. nickel steel No. 3450 is hardly affected by the previous removal of its scale.

In view of the large and growing use of welding processes, this specimen was prepared with an electrically-welded joint across its middle, the weld being autogeneous to avoid as far as possible any electrolytic effects in the subsequent corrosion, although some difference in composition of the weld-metal might be expected. The corrosion on the weld-metal is no more pronounced than on the parent

steel, the chief feature being a somewhat localized corrosion in the region adjoining the weld; this corrosion is not, however, any deeper than at some other places on the steel, and it would appear that, from the point of view of corrosion, the welded steel can be used with the same safety as the homogeneous material.

Conclusions.—A third series of specimens has been put out for exposure and these tests have not yet been completed, but it is possible to draw some conclusions from the results reported above.

The fact which appears to be one of the broadest conclusions from the Sea-Action Committee's research, namely, that the advantages to be anticipated from the use of alloy steels in underwater use are not so marked as under aerial or half-tide conditions, is confirmed in the present series of trials where the range of materials tested has been extended. With steels of the modern "non-rusting" type, containing high percentages of nickel and chromium, the actual total wastage by corrosion can be greatly minimized, but, as with the high-chromium steel included in the Committee's research, these steels have a marked propensity to localized pitting, which might be expected to restrict seriously their life in actual service under water and in some cases might make their use dangerous.

These conclusions may appear different from well-authenticated cases where specimens of the 18-per-cent. chromium—8-per-cent. nickel type of steel have behaved admirably when, for example, they have been attached to the hull of a ship during a long voyage; the protection afforded by electrolytic contact with ordinary steel or other metal was presumably avoided in such cases. It is conceivable that, in these cases, the rapid movement of the sea-water in contact with the steel may have a favourable influence on its behaviour. Whatever view is adopted, the washing action of the water over the surface of the steel would certainly be expected to modify drastically the conditions which promote pitting. In the present phase of research, new methods have been adopted to enable a more intimate examination to be made of the separate factors operating in corrosion; and it can therefore be understood that there is an abundance of work to be done in investigating even a few metals in their elementary forms. It is to be hoped, however, that sufficient progress will have been made before long that problems such as the one now mentioned, and concerning points of importance in the development of non-rusting steels, may be examined. In the meantime, it may well be that, although the use of steels containing high percentages of chromium cannot be recommended for stationary structures in contact with sea-water, such steels are able to give good accounts of themselves in circumstances more favourable to their peculiarities.

It should be mentioned that the Sea-Action Committee, after careful consideration, preferred studiously to avoid the introduction of these and other steels in the same general class which have been developed since the commencement of their research in 1916, as they desired to devote their activities to those selected in the original programme. The best and most reliable material, among those available at the present stage of development of alloy steels, appears to be the steel containing about 36 per cent. of nickel, the present results for this material fully confirming those obtained in the Committee's research. The question as to the possibility of making use of the excellent corrosion-resisting properties of this steel in marine structure has been discussed on page 42 of this Paper.

Final Remarks and Acknowledgments.

As may be seen from a study of the Paper, the subject of corrosion is most complex, and much still remains to be done before its effects can be adequately understood and dealt with. Now that the united efforts of metallurgists, engineers, chemists, and scientific workers have been organized into a world campaign against corrosion, it may be expected that their work will go far towards finding a solution of the problem.

The Authors would like to take this opportunity of thanking the Board of the Department of Scientific and Industrial Research for their financial assistance in this important work, and to make particular mention of the friendly co-operation of the secretaries of the Department, first Sir Frank Heath, G.B.E., K.C.B., and now Sir Frank Smith, K.B.E., F.R.S. They would also like to express their cordial thanks for the assistance and advice of Mr. W. J. Dawson, Metallurgical Director, Mr. T. G. Elliot, F.I.C., head of the chemical laboratories, and Mr. A. Stevenson, all of Messrs. Hadfields, Ltd., Sheffield.

The Paper is accompanied by nine sheets of drawings and fourteen photographs, from some of which the Figures in the text and the two half-tone page-plates have been prepared, and by the following Appendixes.

APPENDIXES.

APPENDIX I.

Committees on Corrosion.

In Great Britain.—The Committee of The Institution of Civil Engineers on the Deterioration of Structures Exposed to Sea-Action. This Committee was formed in 1916 and its researches on Corrosion are concerned with iron and steel.

The Corrosion Research Committee of the Institute of Metals, formed in 1910, and working in conjunction with Research Sub-Committee No. 46 of the British Non-Ferrous Metals Research Association.

Research Sub-Committee No. 4 on Atmospheric Corrosion, of the British Non-Ferrous Metals Research Association. This Committee commenced its work in 1920.

The Corrosion of Metals Research Committee of the Department of Scientific and Industrial Research. This Committee was established in 1924.

The Joint Corrosion Committee of the Iron and Steel Institute and the National Federation of Iron and Steel Manufacturers, formed in 1928.

The Corrosion of Pipes Sub-Committee of the Institution of Gas Engineers, formed in 1928.

In the United States.—Committees of the American Society for Testing Materials, as follows:—A.5, on the Corrosion of Iron and Steel, established in 1915; B.3, for Non-Ferrous Metals; A.10, on Methods of Corrosion-Testing for Iron-Chromium, Iron-Chromium-Nickel and related alloys; B.6, on Exposure and Corrosion-Tests for Die-Cast Metals; B.7, on Methods of Protection of Light Metals against Corrosion. As a means of contact between these five committees and for the co-ordination of their activities, there is also a special Co-ordinating Committee on Corrosion.

In Germany.—A National Committee for the Study of the Protection of Metals (Reichsausschuss für Metallschutz).

The Corrosion Committee of the D.V.G.W., in which are co-operating the Verein deutscher Ingenieure, the Verein deutscher Eisenhuttenleute, the Deutsche Gesellschaft für Metallkunde, and the Verein deutscher Chemiker.

In Holland.—The Corrosion Committee instituted by the Research Institute for Materials (Stichting voor Materiaalonderzoek), for investigating the phenomena of corrosion, and for finding means of arresting it. This Committee was formed in 1931.

In Russia.—The Research Institute of Shipbuilding at Leningrad is conducting researches on the corrosion of marine structural steels.

In Sweden.—The Swedish Commission on Corrosion of Metals. (Studienämnden för Korrosionsskyddsfragor.)

In Switzerland.—The Corrosion Committee of the Societé Suisse de l'Industrie de Gaz et des Eaux.

It is not necessary in this Paper to attempt to deal fully with the work of these Committees. Some of them, as will appear, are concerned with quite specialized aspects of corrosion. The reports of the committees are available for reference.

It should, however, be mentioned that the establishment in 1910 of the Committee of the Institute of Metals was the earliest example of a serious attempt by a technical society to solve a practical problem of corrosion. It was brought into being through the difficulties experienced in the corrosion of

condenser tubes, the complex causes of which have been sufficient to occupy the continuous attention of the Committee ever since that date.

The Committee A.S. has paid special attention to copper-bearing steel, the exposure tests being planned on an extensive scale. Reports have been issued annually.

The British Joint Committee of the Iron and Steel Institute and the National Federation of Iron and Steel Manufacturers, formed in 1928 with Dr. W. H. Hatfield, F.R.S., as Chairman, has for its principal objective the reduction of the wastage by corrosion of iron and steel, whether by improvements in the material itself, or by methods of protection. For this purpose their investigations are intended eventually to explore every aspect of the corrosion of iron and steel. They have already issued three reports which contain a great deal of information for those studying corrosion, both in its practical and laboratory aspects. The initial programme of the Committee's field tests is described, with very complete details of the manufacture of the steels tested, and the preparation of the test specimens, as well as reports on the progress of their corrosion during exposure under a great variety of conditions. This first programme is concerned with mild steel and the effect of additions of small amounts of copper up to 0·5 per cent.

Special reference should also be made to the Dutch Committee which was formed in 1931, and which includes among its members, under the Chairmanship of Dr. N. J. Muller, Chief Superintendent Engineer of the Royal Packet Navigation Company, engineers and others occupying important positions in Holland. The work of the Committee is divided among several subcommittees, including No. IV, dealing with the atmospheric corrosion of iron, and No. V with the corrosion of steel in sea-water. The scientific investigator is Dr. H. van der Veen.

In November, 1933, several members of this Committee paid a welcome visit to Great Britain, for the purpose of informing themselves of what is being done here in the way of investigations on corrosion, and to discuss problems of mutual interest with those engaged in this work.

APPENDIX II.

TABLE I.—TEST SPECIMENS OF CHROMIUM STEEL—SOFT GRADE.
Rolled bars 3 inches by $\frac{3}{8}$ inch by 24 inches long.

Classification-letter.	Individual numbers of bars.	Heat-treatment.	Condition of surface.
S.	1 to 3.	As rolled.	With scale.
T.	1 to 3.	800° C., furnace.	With scale.
U.	1 to 3.	800° C., furnace.	Pickled.
V.	1 to 3.	800° C., furnace.	Polished.
W.	1 to 3.	900° C., water. 400° C., water.	With scale.
X.	1 to 3.	900° C., water. 400° C., water.	Polished.

Table II.—Particulars of Manufacture of the Various Ferrous Materials used in the Committee's Research. Also particulars of the numbers of the bars prepared from them, and their allocation.
(The full analyses of these ferrous materials are shown in Tables III, IV, V, and VI.)

Classification letter.	Material.	Manufacture. Process.	Ingots.	Cogging the ingots into billets under the press.	Rolling bars from the billets.	Allocation of the test bars. Purpose for which used.	Bars exposed. No. of bars.	Individual numbers.	No. of spares.
		Steels made by Hadfields, Ltd.							
A.	Medium carbon steel, low S. and P.	Basic open hearth.	3 14-inch.	Two 14-inch ingots heated to 1170° C. and forged into 3-inch square billets.	Heated to 1,100° C., cogged down at mill to $3\frac{1}{2}$ inches by 1 inch. Reheated to 1,080° C. and rolled into bars.	Exposure at various ports.	39	1-39	46
E.						Exposure at various ports, mechanical tests (untreated and treated). Tensile bar for sea immersion.	66	1-63, 81-82, 83	32
B.	Mild steel, low Mn, high S. and P.	Converter.	4 11-inch, 1 9-inch.	One 11-inch ingot heated to 1,170° C. and cogged into 3-inch square billets.	Heated to 1,170° C., cogged down at mill to $3\frac{1}{2}$ inches by 1 inch. Reheated to 1,080° C. and rolled into bars.	,,	66	1-63, 81-82, 83	23
C.	Mild steel with 0·7 per cent. manganese.	Acid open hearth.	4 11-inch, 1 10-inch, 1 5-inch.	Two 11-inch ingots heated to 1,210° C. and forged into 3-inch square billets.	Rolled as above into 31/32 inch by 33/64 inch.	,,	75	1-75	8
F.							42	1-39, 81 and 82, 84	45
D.	0·40-per-cent. carbon steel.	Basic open hearth.	4 11-inch, 1 8-inch.	One 11-inch ingot heated to 1,235° C., rolled into 3-inch square billets 60 lbs. each.	Heated to 1,085° C. and rolled into bars.	,,	54	1-51, 81 and 82, 83	32
G.	Mild steel with $\frac{1}{2}$ per cent. copper.	Electric arc.	4 11-inch.	One 11-inch ingot heated to 1,180° C. and forged into 3-inch square billets.	Heated to 1,085° C., cogged down at mill to $3\frac{1}{2}$ inches by 1 inch. Reheated to 1,080° C. and rolled into bars.	,,	42	1-39, 81 and 83, 84	47
H.	Mild steel with 2 per cent. copper.	,,	4 11-inch.	Two 11-inch ingots heated to 1,080° C. and forged into 3-inch square billets, and then reheated to 1,000° C. and cogged down to $3\frac{1}{2}$ inches by 1 inch.	Reheated to 1,000° C. and rolled into bars.	,,	42	1-39, 81 and 82, 83	41
J.	Chromium steel, hard grade.	,,	1 9-inch, 1 8-inch, 1 6-inch, 1 9·2-inch.	One 9-inch ingot heated to 1,100° C. and forged down to 4-inch square billets. Reheated to 1,130° C. and further forged into 7 inches diameter. Again reheated to 1,100° C. and pressed to 4-inch square billets.	Heated to 1,130° C. and forged into 3 inches square at the mill, then reheated to 1,120° C. and rolled into bars.	,,	87	1-80, 83-86, 81 and 82, 91	1
,,	,, Additional ground and polished bars.					Exposure at Plymouth only.	4	87-90	
S.	Chromium steel, soft grade.	,,	4 6-inch, 4 11-inch, 1 15-inch.	One 6-inch ingot heated to 1,150/1,200° C. forged to $3\frac{1}{2}$ inches by 1 inch.	Heated to 1,180/1,220° C. and rolled to 3 inches by $\frac{3}{8}$ inch section.	Exposure at Plymouth only. Mechanical tests.	4	1-3, 4	
T.						,,	4	1-3, 4	1
U.						Exposure at Plymouth only.	3	1-3	2
V.						,,	3	1-3	3
W.						Exposure at Plymouth only. Mechanical tests.	4	1-3, 4	1
X.						Exposure at Plymouth only.	3	1-3	
K.	$3\frac{1}{2}$-per-cent. nickel steel.	Acid open hearth.	4 11-inch, 1 10-ton, 1 5-ton.	One 11-inch ingot heated to 1,235° C. and rolled into 3-inch square billets.	Heated to 1,085° C. and rolled into bars.	Exposure at various ports. Mechanical tests (untreated and treated). Tensile bar for sea immersion.	42	1-39, 81 and 82, 84	43
L.	36-per-cent. nickel steel.	Crucible.	16 4-inch.	Twelve 4-inch ingots heated to 970° C., cogged into $3\frac{1}{2}$ inches by 1 inch.	Heated to 970° C. and rolled into 3 inches by $\frac{1}{2}$ inch section.	,,	78	1-75, 81 and 82, 83	8
		Materials purchased. Obtained from :—							
M.	Ingot iron.	The Shelton Iron and Steel Co., Ltd.				Tensile bar for sea immersion.	54	1-51, 81 and 82, 83	59
N.	Wrought iron.	The Low Moor Iron Co., Ltd.				,,	84	1-80, and 83, 81 and 82, 84	21
P.	Swedish charcoal iron.	(LSS) Brand, Upsala, Sweden.				,,	66	1-63, 81 and 82, 83	17
Q.	Hot-blast cast iron.	The Lilleshall Co.				Exposure at various ports. Mechanical tests (untreated only). Tensile bar for sea immersion.	65	1-63, 81, 83	24
R.	Cold-blast cast iron.	,,				,,	53	1-51, 81, 83	36
						Total . . .	980		490*

Note.—All but A and E top-poured. Ingots fed by Hadfield system using slag and charcoal, except those for material "L," which were fed through a fireclay nozzle.

Bars for exposure	937 (excluding spares)	Grand Total . 980
Tensile bars for exposure	14	
Bars for mechanical tests	29	

* Also 191 scrap pieces.

TABLE III.—SECTION III.—SPECIAL STEELS. Report on Mechanical Tests.

The test-pieces for this series were cut from bars 24 inches by 3 inches by ½ inch, No. 4 for materials S, T, U, V, W and X, and Nos. 81 and 82 for the others. For positions and dimensions of test-pieces, see *Fig. 1* of the Paper, "Corrosion of Ferrous Metals," Minutes of Proceedings, Inst. C.E., vol. ccxiv (1922), p. 83.

Type of material.	Classification letter.	Analysis: per cent. C.	Si.	S.	P.	Mn.	Cr.	Ni.	Cu.	Treatment.*	Section.	Mechanical tests. Tensile tests. Yield-point: tons per sq. in.	Max. stress: tons per sq. in.	Elong. on 8 ins.: per cent.	Redn. of area: per cent.	Fracture.	Frémont shock test. Nicked. Kg.-m.	Nicked. Angle: degrees.	Unnicked. Kg.-m.	Unnicked. Angle: degrees.	Brinell test on Frémont specimen.	Izod shock test. Ft.-lbs.	Angle: degrees.
										(A) Untreated Material.													
Mild steel with ½ per cent. copper.	G.	0·21	0·14	0·043	0·046	0·94	—	—	0·63	As rolled.	Long.	25·5	37·8	24·75	55·54	Granular.	18·6	87	43·0	132, unb.	152	62	25
		—	—	—	—	0·95	—	—	0·64		Trans.	—	—	—	—	—	3·7	6	23·0	80	153	—	—
Mild steel with 2 per cent. copper.	H.	0·22	0·14	0·033	0·041	0·90	—	—	2·18	As rolled.	Long.	31·1	55·5	14·25	39·91	Granular. Traces v. f. cryst.	5·2	8	39·5	95	250	18	6
		0·23	—	0·035	0·043	0·92	—	—	2·19		Trans.	—	—	—	—	—	4·0	5	18·4	41	256	—	—
3½-per-cent. nickel steel.	K.	0·31	0·18	0·029	0·038	0·54	—	3·75	—	As rolled.	Long.	25·5	39·5	21·75	50·53	Granular.	7·0	20	40·6	116	170	19	7
											Trans.	—	—	—	—	—	4·6	12	26·0	80	176	—	—
36-per-cent. nickel steel.	L.	0·09	0·09	—	—	0·77	—	36·67	—	As rolled.	Long.	23·3	38·9	24·50	53·38	Lt. grey, fibrous, silky.	20·7	95	40·7	128, unb.	161	107	58, unb.
		0·15	—	—	—	0·97	—	36·45	—		Trans.	—	—	—	—	—	5·5	18	27·1	83	179	—	—
Chromium steel, hard grade.	J.	0·36	0·22	—	—	0·13	13·57	—	—	600° C. furnace.	Long.	33·9	64·8	14·12	39·21	Dark grey.	5·7	7	26·6	53	296	26	10
											Trans.	—	—	—	—	—	3·3	2	14·6	27	294	—	—
Chromium steel, soft grade.	S.	0·12	0·53	0·042	0·015	0·21	11·90	—	—	As rolled.	Long.	50·9	67·9	11·25	36·70	Granular. Partly sheared fracture.	11·7	49	26·0	44	338	120†	48, unb.
											Trans.	—	—	—	—	—	2·9	6	14·1	23	341	—	—
										(B) Treated Material.													
Mild steel with ½ per cent. copper.	G.	0·21	0·14	0·043	0·046	0·94	—	—	0·63	850° C. water. 650° C. water.	Long.	31·9	40·6	17·75	59·03	Silky, fibrous.	32·2	104	57·0	143, unb.	180	106	60, unb.
		—	—	—	-	0·95	-		0·64		Trans.	—	—	—	—		10·4	26	41·0	114	179	—	—
Mild steel with 2 per cent. copper.	H.	0·22	0·14	0·033	0·041	0·90	—	—	2·18	850° C. water. 700° C. water.	Long.	47·8	52·9	13·75	55·83	Silky, fibrous.	23·8	89	49·4	121, unb.	236	92	62, unb.
		0·23	—	0·035	—	0·92	-	—	2·19		Trans.	—	—	—	—		7·6	14	29·0	80	238	—	—
3½-per-cent. nickel steel.	K.	0·31	0·18	0·029	0·038	0·54	—	3·75	—	850° C. oil. 650° C. water.	Long.	42·8	49·9	17·50	56·11	Granular.	18·0	68	50·0	120	234	68	40
											Trans.	—	—	—	—		8·2	16	29·6	68	240	—	—
36-per-cent. nickel steel.	L.	0·09	0·09	—	—	0·77	—	36·67	—	940° C. water.	Long.	18·9	34·0	35·62	60·26	Silky, fibrous.	27·[illegible]	154, unb.	36·9	134, unb.	130	102	60, unb.
		0·15	—	—	—	0·97	—	36·45	—		Trans.	—	—	—	—		13·2	58	34·2	123	128	—	—
Chromium steel, hard grade.	J.	0·36	0·22	—	—	0·13	13·57	—	—	875° C. water. 750° C. water.	Long.	34·4	50·2	16·37	57·11	Dk. grey, granular.	11·[illegible]	31	48·3	121, unb.	214	44	20
											Trans.	—	—	—	—		6·[illegible]	16	25·1	64	218	- -	—
Chromium steel, soft grade.	T, U and V.	0·12	0·53	0·042	0·015	0·21	11·90	—	—	800° C. furnace.	Long.	21·6	34·3	24·8	58·75	Granular.	21·[illegible]	113	44·0	143, unb.	155	70[2]	50, unb.
											Trans.	—	—	—			8·4	36	48·0	135, unb.	166	—	—
Chromium steel, soft grade.	W and X.	0·12	0·53	0·042	0·015	0·21	11·90	—	—	900° C. water. 400° C. water.	Long.	69·4	81·0	10·38	49·72	Granular.	5·[illegible]	11	33·8	53	395	6[2]	3
											Trans.	—	—	—	—	—	3·[illegible]	5	24·4	40	406	—	—

The specimens S, T, U, V, W and X represent the composition and mechanical properties of the additional test-bars forwarded to the Committee in 1924.

* Under this column is given the heat-treatment received by each specimen; for example, "970° C. water, 560° C. water," means that the specimen was heated until it had attained a temperature of 970° C. and was then quenched in water at about 20° C., being afte heated to a temperature of 560° C., and then quenched again in water at 20° C.

† Owing to the thickness of the material being only ⅜-inch, the standard Izod section of 10 by 10 mm. could not quite be obtained. The standard length, width and depth of notch were adhered to, the cross section behind the notch being 7½ by 10 mm. instead of 8 by

TABLE IV.—SECTION I.—IRONS (ROLLED AND FORGED).

Report on Mechanical Tests.

The test-pieces for this series were cut from bars 24 inches by 3 inches by ½ inch, Nos. 81 and 82 for each material.
For positions and dimensions of test-pieces, see *Fig. 1* of the Paper, "Corrosion of Ferrous Metals," Minutes of Proceedings, Inst. C.E., vol. ccxiv (1922), p. 83.

Type of material.	Classification letter.	Analysis: per cent.								Treatment.[1]	Section.	Mechanical tests.												
												Tensile tests.					Frémont shock test.					Izod shock test.		
																	Nicked.		Unnicked.		Brinell test on Frémont specimen.			
		C.	Si.	S.	P.	Mn.	Cr.	Ni.	Cu.			Yield-point: tons per sq. in.	Max. stress: tons per sq. in.	Elong. on 8 ins.: per cent.	Redn. of area: per cent.	Fracture.	Kg.-m.	Angle: degrees.	Kg.-m.	Angle: degrees.		Ft.-lbs.	Angle: degrees.	Brinell test on Izod specimen.
(A) Untreated Material. Bar No. 81 of each material.																								
Wrought iron.	N.	0·02 0·01	0·14 0·15	0·013 0·012	0·113 0·114	Tr. 0·03		No. 1 bar No. 2 bar		As rolled.	Long.	14·2	22·2	33·12	46·61	Coarse fibrous.	4·8	16	22·3	107	98	22½	15	106
											Trans.	—	—	—	—		2·6	3	4·2	14	102	—	—	—
Swedish charcoal iron.	P.	0·02 0·04	0·03 —	0·019 —	0·022 —	0·01 0·03		No. 1 bar No. 2 bar		As rolled.	Long.	12·2	20·4	34·00	62·67	Coarse fibrous.	21·6	110	32·3	135, unb.	88	65	61, unb.	87
											Trans.	—	—	—	—		4·0	18	10·4	54	80	—	—	—
Ingot iron.	M.	0·03 0·04	0·03 0·04	0·032 0·033	0·016 0·019	0·06 0·07		No. 1 bar No. 2 bar		As rolled.	Long.	14·4	22·2	27·35	64·47	Coarse fibrous.	10·0	29	34·6	140, unb.	90	16	8	96
											Trans.	—	—	—	—	—	2·6	2	34·8	141, unb.	94	—	—	—
(B) Treated Material. Bar No. 82 of each material.																								
Wrought iron.	N.	0·02 0·01	0·14 0·15	0·013 0·012	0·113 0·114	Tr. 0·03		No. 1 bar No. 2 bar		970° C. water 560° C. water	Long.	18·2	26·7	15·50	39·13	Fibrous.	11·5	64	33·6	143, unb.	114	33	18	112
											Trans.	—	—	—	—	—	5·8	4	4·6	5	132	—	—	—
Swedish charcoal iron.	P.	0·02 0·04	0·03 —	0·019 —	0·022 —	0·01 0·03		No. 1 bar No. 2 bar		970° C. water 560° C. water	Long.	12·6	21·6	29·37	59·84	Coarse fibrous.	21·6	116	36·0	143, unb.	115	71	61, unb.	91
											Trans.	—	—	—	—	—	15·2	78	31·2	100	92	—	—	—
Ingot iron.	M.	0·03 0·04	0·03 0·04	0·032 0·033	0·016 0·019	0·06 0·07		No. 1 bar No. 2 bar		970° C. water 560° C. water	Long.	20·6	27·3	16·25	63·20	Fibrous.	31·3	139, unb.	36·5	141, unb.	103	17½	4	114
											Trans.	—	—	—	—	—	5·5	7	36·5	138, unb.	105	—	—	—

[1] Under this column is given the heat-treatment received by each specimen; for example, "970° C. water, 560° C. water," means that the specimen was heated until it had attained a temperature of 970° C. and was then quenched in water at about 20° C.; afterwards heated to a temperature of 560° C. and quenched again in water at 20° C.

TABLE V.—SECTION II.—CARBON STEELS.

Report on Mechanical Tests.

The test-pieces for this series were cut from bars 24 inches by 3 inches by ½ inch, Nos. 81 and 82 for each material.
For positions and dimensions of test-pieces, see *Fig. 1* of the Paper, "Corrosion of Ferrous Metals," Minutes of Proceedings, Inst. C.E., vol. ccxiv (1922), p. 83.

Type of material.	Classification letter.	Analysis: per cent.								Treatment.[1]	Section.	Mechanical tests.												
												Tensile tests.					Frèmont shock test.					Izod shock test.		
																	Nicked.		Unnicked.		Brinell test on Frèmont specimen.			
		C.	Si.	S.	P.	Mn.	Cr.	Ni.	Cu.			Yield-point: tons per sq. in.	Max. stress: tons per sq. in.	Elong. on 8 ins.: per cent.	Redn. of area: per cent.	Fracture.	Kg.-m.	Angle: degrees.	Kg.-m.	Angle: degrees.		Ft.-lbs.	Angle: degrees.	Brinell test on Izod specimen.
(A) Untreated Material. Bar No. 81 of each material.																								
Mild steel with low Mn. and high S. and P.	B.	0·21	0·17	0·090	0·060	0·33	—	—	—	As rolled.	Long.	22·2	34·2	23·12	47·66	Granular.	16·1	82	39·4	135, unb.	132	48	28	146
		0·22	—	0·115	0·073	0·35	—	—	—		Trans.	—	—	—	—	—	4·4	11	20·5	78	138	—	—	—
Mild steel with 0·7 per cent. Mn.	F.	0·23	0·16	0·061	0·048	0·68	—	—	—	As rolled.	Long.	22·2	33·5	26·25	57·12	Silky fibrous.	17·3	81	39·4	139, unb.	145	82	62, unb.	144
		0·25	—	0·064	—	—	—	—	—		Trans.	—	—	—	—	—	6·5	22	26·0	83	140	—	—	—
Medium carbon steel with low S. and P.	E.	0·34	0·20	0·024	0·027	0·71	—	—	—	As rolled.	Long.	23·4	40·0	26·25	51·77	Granular.	8·3	23	43·7	135, unb.	161	38	17	168
		0·35	—	0·026	—	0·72	—	—	—		Trans.	—	—	—	—	—	5·8	14	35·4	112	165	—	—	—
0·40 per cent. carbon steel.	D.	0·40	0·20	0·049	0·045	0·85	—	—	—	As rolled.	Long.	23·3	41·1	21·00	45·23	Granular.	7·2	17	43·2	129	169	30	11	178
											Trans.	—	—	—	—	—	3·9	7	20·4	58	169	—	—	—
(B) Treated Material. Bar No. 82 of each material.																								
Mild steel with low Mn. and high S. & P.	B.	0·21	0·17	0·090	0·060	0·33	—	—	—	900° C. water	Long.	25·0	34·1	23·75	56·94	Silky granular.	29·6	156, unb.	43·0	136, unb.	149	92½	60, unb.	147
		0·22	—	0·115	0·073	0·35	—	—	—	650° C. water	Trans.	—	—	—	—	—	7·7	19	33·1	113	152	—	—	—
Mild steel with 0·7 per cent. Mn.	F.	0·23	0·16	0·061	0·048	0·68	—	—	—	900° C. water	Long.	30·5	42·8	15·50	54·34	Granular.	18·7	77	47·1	125, unb.	197	87	61, unb.	200
		0·25	—	0·064	—	—	—	—	—	700° C. water	Trans.	—	—	—	—	—	8·0	19	30·2	84	201	—	—	—
Medium carbon steel with low S. and P.	E.	0·34	0·20	0·024	0·027	0·71	—	—	—	900° C. water	Long.	27·8	44·1	19·12	57·51	Fibrous.	29·0	100	47·1	129, unb.	186	86	36	202
		0·35	—	0·026	—	0·72	—	—	—	650° C. water	Trans.	—	—	—	—	—	12·2	24	41·9	123	189	—	—	—
0·40 per cent. carbon steel.	D.	0·40	0·20	0·049	0·045	0·85	—		—	900° C. water	Long.	31·4	46·4	17·37	56·67	Silky granular.	25·3	95	47·6	128, unb.	194	106	60, unb.	191
										650° C. water	Trans.	—	—	—	—	—	6·9	17	33·7	91	183	—	—	—

[1] Under this column is given the heat treatment received by each specimen; for example, "970° C. water, 560° C. water," means that the specimen was heated until it had attained a temperature of 970° C., and was then quenched in water at about 20° C.; afterwards heated to a temperature of 560° C. and quenched again in water at 20° C.

Table VII.—Section VI.—Chromium Steel—Soft Grade.

Tensile Tests.—Detailed Figures of Elongation along Whole Length of Test-Bar.

The bars before testing were marked off with centre dots every ½ inch along the 8-inch gauge-length. After testing, the amount of elongation in every inch along the bar was measured. In every case the measurements were commenced from the end farthest away from fracture.

Full details of analysis, tensile tests, and heat-treatments of these specimens will be found in Table I, p. 70.

Classification letter.	Treatment.	Elongation in inches.								Total elongation in 8 inches.	
		First inch.	Second inch.	Third inch.	Fourth inch.	Fifth inch.	Sixth inch.	Seventh inch.	Eighth inch.	Inches.	Per cent.
		Untreated material.									
S.	As rolled.	0·04	0·06	0·06	0·06	0·10	0·34 (Frac.)	0·16	0·06	0·90	11·25
		Treated material.									
T, U, V.	800° C., furnace.	0·14	0·17	0·18	0·18	0·16	0·24	0·68 (Frac.)	0·24	1·99	24·88
W, X.	900° C., water. 400° C., water.	0·07	0·08	0·06	0·06	0·05	0·06	0·35 (Frac.)	0·10	0·83	10·38

Table VIII.—Section VI.—Chromium Steel—Soft Grade.

Report on Microstructures.

Analysis: per cent.	C.	Si.	S.	P.	Mn.	Cr.	Fe. (by difference)
	0·12	0·53	0·042	0·015	0·21	11·90	87·183

The test-pieces for this series were cut from the same bars 24 inches by 3 inches by $\frac{3}{8}$ inch as the mechanical tests shown in Table II. For position of test-pieces, see *Fig. 1* of the Paper "Corrosion of Ferrous Metals," Minutes of Proceedings, Inst. C.E., vol. ccxiv (1922), p. 83. The reports on the microstructures of the remaining materials included in the research are contained in Tables XII to XIV of the same Paper.

Type of material.	Classification letter.	Reference No. (*Figs. 1* to *8*, facing p. 18).	Section.	Magnification.	Description of Microstructure.
		(A) Untreated material (as rolled).			
Non-rusting steel—soft grade.	S.	5,250 5,251	Long. Trans.	×100 ×600	An acicular troosto-martensitic ground-mass in which are numerous parallel bands of small grains of chromium-ferrite.
		(B) Treated material (800° C., cooled in furnace).			
Do.	T.U.V.	3,841 3,842	Long. Trans.	×100 ×600	Small grains of chromium-ferrite with small particles of free carbide lying chiefly in the grain boundaries.
		(C) Treated material (900° C. water; 400° C. water).			
Do.	W, X.	3,843 3,844	Long. Trans.	×100 ×600	An acicular martensitic ground-mass on which are bands composed of grains of chromium-ferrite. Tiny particles of free carbide also are irregularly distributed throughout the ground-mass.

APPENDIX III.

TABLE IX.—INFLUENCE OF PREVIOUS REMOVAL OF SCALE ON CARBON STEELS.
General corrosion.

			Wastage: millimetres.			
			Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*						
E.	Medium carbon steel, low S. and P.	With scale.	0·108	0·346	0·475	0·310
A.		Without scale.	0·078	0·220	0·539	0·280
F.	Mild steel 0·7 per cent. Mn.	With scale.	0·132	0·424	0·475	0·344
C.		Without scale.	0·076	0·274	0·466	0·272
E & F.	Average.	With scale.	0·120	0·385	0·475	0·327
A & C.	,,	Without scale.	0·077	0·247	0·502	0·276
(2) *Auckland.*						
E.	Medium carbon steel, low S. and P.	With scale.	0·218	0·151	0·437	0·270
A.		Without scale.	0·174	0·274	0·584	0·345
F.	Mild steel, 0·7 per cent. Mn.	With scale.	0·192	0·158	0·445	0·265
C.		Without scale.	0·210	0·296	0·495	0·334
E & F.	Average.	With scale.	0·205	0·154	0·441	0·267
A & C.	,,	Without scale.	0·192	0·285	0·539	0·339
(3) *Plymouth.*						
E.	Medium carbon steel, low S. and P.	With scale.	0·808	0·305	0·465	0·526
A.		Without scale.	0·258	0·574	0·530	0·454
F.	Mild steel, 0·7 per cent. Mn.	With scale.	1·005	0·346	0·526	0·626
C.		Without scale.	0·358	0·585	0·642	0·528
E & F.	Average.	With scale.	0·906	0·325	0·495	0·576
A & C.	,,	Without scale.	0·308	0·579	0·586	0·491
(4) *Colombo.*						
E.	Medium carbon steel, low S. and P.	With scale.	2·000	1·650	0·589	1·418
A.		Without scale.	2·200	1·617	0·725	1·513
F.	Mild steel, 0·7 per cent. Mn.	With scale.	1·919	1·153	0·504	1·193
C.		Without scale.	2·090	0·988	0·563	1·215
E & F.	Average.	With scale.	1·959	1·401	0·546	1·303
A & C.	,,	Without scale.	2·145	1·302	0·644	1·364
(5) *All four ports.*						
E.	Medium carbon steel, low S. and P.	With scale.	0·783	0·613	0·491	0·630
A.		Without scale.	0·677	0·671	0·594	0·648
F.	Mild steel, 0·7 per cent. Mn.	With scale.	0·812	0·520	0·487	0·607
C.		Without scale.	0·683	0·536	0·541	0·587
E & F.	Average.	With scale.	0·797	0·566	0·489	0·618
A & C.	,,	Without scale.	0·680	0·603	0·567	0·617

TABLE X.—INFLUENCE OF PREVIOUS REMOVAL OF SCALE ON CARBON STEEL.

Pitting.

			Depth of deepest pit: millimetres.			
			Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*						
E.	Medium carbon steel, low S. and P.	With scale.	0·43	5·10	1·86	2·46
A.		Without scale.	nil	2·52	0·76	1·09
F.	Mild steel, 0·7 per cent. Mn.	With scale.	nil	3·10	1·52	1·54
C.		Without scale.	0·425	1·02	1·40	0·95
E & F.	Average.	With scale.	0·21	4·10	1·69	2·00
A & C.	,,	Without scale.	0·21	1·77	1·08	1·02
(2) *Plymouth.*						
E.	Medium carbon steel, low S. and P.	With scale.	0·65	1·73	1·72	1·37
A.		Without scale.	0·29	1·13	1·34	0·92
F.	Mild steel, 0·7 per cent. Mn.	With scale.	0·59	1·22	3·13	1·65
C.		Without scale.	0·53	0·99	0·69	0·74
E & F.	Average.	With scale.	0·62	1·47	2·42	1·51
A & C.	,,	Without scale.	0·41	1·06	1·01	0·83
(3) *Auckland.*						
E.	Medium carbon steel, low S. and P.	With scale.	1·67	0·59	2·79	1·68
A.		Without scale.	0·63	0·55	0·62	0·60
F.	Mild steel, 0·7 per cent. Mn.	With scale.	0·97	0·39	2·32	1·23
C.		Without scale.	nil	nil	0·44	0·15
E & F.	Average.	With scale.	1·32	0·49	2·55	1·45
A & C.	,,	Without scale.	0·31	0·27	0·53	0·37
(4) *Colombo.*						
E.	Medium carbon steel, low S. and P.	With scale.	1·37	4·56	2·76	2·90
A.		Without scale.	2·17	1·97	1·17	1·77
F.	Mild steel, 0·7 per cent. Mn.	With scale.	4·56	5·30	3·15	4·34
C.		Without scale.	1·68	0·74	1·03	1·15
E & F.	Average.	With scale.	2·96	4·93	2·93	3·62
A & C.	,,	Without scale.	1·92	1·36	1·10	1·46
(5) *All four ports.*						
E.	Medium carbon steel, low S. and P.	With scale.	1·03	3·00	2·28	2·10
A.		Without scale.	0·77	1·54	0·97	1·09
F.	Mild steel, 0·7 per cent. Mn.	With scale.	1·53	2·50	2·58	2·19
C.		Without scale.	0·66	0·69	0·89	0·75
E & F.	Average.	With scale.	1·28	2·75	2·43	2·14
A & C.	,,	Without scale.	0·72	1·11	0·93	0·92

TABLE XI.—ROLLED IRONS COMPARED WITH ORDINARY STEELS.

General corrosion.

		Wastage : millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
N, P, M.	Rolled irons.	0·151	0·313	0·550	0·338
B, D, E, F.	Ordinary steels.	0·130	0·463	0·476	0·357
Average		0·140	0·388	0·513	0·347
(2) *Auckland.*					
N, P, M.	Rolled irons.	0·349	0·230	0·409	0·329
B, D, E, F.	Ordinary steels.	0·251	0·143	0·452	0·282
Average		0·300	0·186	0·430	0·305
(3) *Plymouth.*					
N, P, M.	Rolled irons.	0·929	0·482	0·486	0·632
B, D, E, F.	Ordinary steels.	0·973	0·354	0·470	0·599
Average		0·951	0·418	0·478	0·615
(4) *Colombo.*					
N, P, M.	Rolled irons.	1·764	0·836	0·444	1·015
B, D, E, F.	Ordinary steels.	1·904	1·193	0·546	1·214
Average		1·834	1·014	0·495	1·114
(5) *All four ports.*					
N, P, M.	Rolled irons.	0·797	0·465	0·472	0·579
B, D, E, F.	Ordinary steels.	0·814	0·538	0·486	0·613
Average		0·805	0·501	0·479	0·596

TABLE XII.—ROLLED IRONS COMPARED WITH ORDINARY STEELS.

Pitting.

		Depth of deepest pit: millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
N, P, M.	Rolled irons.	nil	3·20	0·54	1·25
B, D, E, F.	Ordinary steels.	0·32	3·53	1·54	1·79
Average		0·16	3·36	1·04	1·52
(2) *Plymouth.*					
N, P, M.	Rolled irons.	0·78	0·94	1·18	0·97
B, D, E, F.	Ordinary steels.	0·58	1·68	2·26	1·51
Average		0·68	1·31	1·72	1·24
(3) *Auckland.*					
N, P, M.	Rolled irons.	1·58	0·07	1·67	1·12
B, D, E, F.	Ordinary steels.	1·49	0·62	2·77	1·62
Average		1·53	0·34	2·22	1·37
(4) *Colombo.*					
N, P, M.	Rolled irons.	1·39	1·24	3·91	2·18
B, D, E, F.	Ordinary steels.	2·26	4·45	3·59	3·43
Average		1·82	2·84	3·75	2·80
(5) *All four ports.*					
N, P, M.	Rolled irons.	0·94	1·36	1·82	1·38
B, D, E, F.	Ordinary steels.	1·17	2·57	2·54	2·09
Average		1·05	1·96	2·18	1·73

TABLE XIII.—THE ROLLED IRONS.

General corrosion.

		Wastage : millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
N. .	Wrought iron.	0·152	0·211	0·540	0·301
P. .	Swedish charcoal iron.	0·152	0·338	0·541	0·344
M. .	Ingot iron.	0·150	0·390	0·570	0·370
Average		0·151	0·313	0·550	0·338
(2) *Auckland.*					
N. .	Wrought iron.	0·281	0·179	0·401	0·287
P. .	Swedish charcoal iron.	0·342	0·238	0·402	0·327
M. .	Ingot iron.	0·425	0·272	0·425	0·374
Average		0·349	0·230	0·409	0·329
(3) *Plymouth.*					
N. .	Wrought iron.	0·729	0·426	0·409	0·521
P. .	Swedish charcoal iron.	1·024	0·499	0·507	0·677
M. .	Ingot iron.	1·034	0·521	0·541	0·699
Average		0·929	0·482	0·486	0·632
(4) *Colombo.*					
N. .	Wrought iron.	1·395	0·809	0·454	0·886
P. .	Swedish charcoal iron.	1·618	0·832	0·388	0·946
M. .	Ingot iron.	2·280	0·868	0·491	1·213
Average		1·764	0·836	0·444	1·015
(5) *All four ports.*					
N. .	Wrought iron.	0·639	0·406	0·451	0·499
P. .	Swedish charcoal iron.	0·784	0·477	0·459	0·573
M. .	Ingot iron.	0·972	0·513	0·507	0·664
Average		0·797	0·465	0·472	0·579

TABLE XIV.—THE ROLLED IRONS.

Pitting.

		Depth of deepest pit: millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions
(1) *Halifax.*					
N. .	Wrought iron.	nil	2·581	1·01	1·20
P. .	Swedish charcoal iron.	nil	4·09	nil	1·36
M. .	Ingot iron.	nil	2·94	0·60	1·18
Average		nil	3·20	0·54	1·25
(2) *Plymouth.*					
N. .	Wrought iron.	1·24	0·74	0·92	0·97
P. .	Swedish charcoal iron.	1·10	0·99	0·75	0·95
M. .	Ingot iron.	nil	1·09	1·88	0·99
Average		0·78	0·94	1·18	0·97
(3) *Auckland.*					
N. .	Wrought iron.	1·12	nil	1·52	0·88
P. .	Swedish charcoal iron.	1·74	nil	1·29	1·01
M. .	Ingot iron.	1·89	0·20	2·19	1·46
Average		1·58	0·07	1·67	1·12
(4) *Colombo.*					
N. .	Wrought iron.	1·23	1·71	1·85	1·60
P. .	Swedish charcoal iron.	1·36	nil	4·85	2·07
M. .	Ingot iron.	1·59	2·00	5·03	2·87
Average		1·39	1·24	3·91	2·18
(5) *All four ports.*					
N. .	Wrought iron.	0·90	1·26	1·32	1·16
P. .	Swedish charcoal iron.	1·05	1·27	1·72	1·35
M. .	Ingot iron.	0·87	1·56	2·42	1·62
Average		0·94	1·36	1·82	1·38

TABLE XV.—THE ORDINARY STEELS.

General corrosion.

		Wastage: millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
D.	0·40 per cent. carbon.	0·105	0·414	0·464	0·328
F.	Mild steel, 0·7 per cent. Mn.	0·138	0·424	0·475	0·344
B.	Mild steel, low Mn., high S. and P.	0·176	0·669	0·491	0·445
E.	Medium carbon, low S. and P.	0·108	0·346	0·475	0·344
Average		0·130	0·463	0·476	0·357
(2) *Auckland.*					
D.	0·40 per cent. carbon.	0·260	0·133	0·483	0·293
F.	Mild steel, 0·7 per cent. Mn.	0·192	0·158	0·445	0·265
B.	Mild steel, low Mn., high S. and P.	0·336	0·131	0·483	0·270
E.	Medium carbon, low S. and P.	0·218	0·151	0·437	0·302
Average		0·251	0·143	0·452	0·282
(3) *Plymouth.*					
D.	0·40 per cent. carbon.	0·983	0·379	0·472	0·612
F.	Mild steel, 0·7 per cent. Mn.	1·005	0·346	0·526	0·626
B.	Mild steel, low Mn., high S. and P.	1·095	0·385	0·416	0·632
E.	Medium carbon, low S. and P.	0·808	0·305	0·465	0·526
Average		0·973	0·354	0·470	0·599
(4) *Colombo.*					
D.	0·40 per cent. carbon.	1·850	1·063	0·581	1·163
F.	Mild steel, 0·7 per cent. Mn.	1·919	1·153	0·504	1·193
B.	Mild steel, low Mn., high S. and P.	1·846	0·905	0·512	1·088
E.	Medium carbon, low S. and P.	2·000	1·650	0·589	1·413
Average		1·904	1·193	0·546	1·214
(5) *All four ports.*					
D.	0·40 per cent. carbon	0·799	0·497	0·500	0·599
F.	Mild steel 0·7 per cent. Mn.	0·812	0·520	0·487	0·607
B.	Mild steel, low Mn., high S. and P.	0·863	0·522	0·466	0·617
E.	Medium carbon, low S. and P.	0·783	0·613	0·491	0·630
Average		0·814	0·538	0·486	0·613

TABLE XVI.—THE ORDINARY STEELS.

Pitting.

	Depth of deepest pit: millimetres.			
	Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*				
B. Mild steel, low Mn., high S. and P.	0·84	1·22	1·81	1·29
E. Medium carbon, low S. and P.	0·43	5·10	1·86	2·46
D. 0·40 per cent. carbon.	nil	4·72	0·96	1·89
F. Mild steel, 0·7 per cent. Mn.	nil	3·10	1·52	1·54
Average	0·32	3·53	1·54	1·79
(2) *Plymouth.*				
B. Mild steel, low Mn., high S. and P.	nil	2·73	2·05	1·59
E. Medium carbon, low S. and P.	0·65	1·73	1·72	1·37
D. 0·40 per cent. carbon.	1·09	1·04	2·14	1·42
F. Mild steel, 0·7 per cent. Mn.	0·59	1·73	3·13	1·65
Average	0·58	1·68	2·26	1·51
(3) *Auckland.*				
B. Mild steel, low Mn., high S. and P.	1·65	0·81	2·28	1·58
E. Medium carbon, low S. and P.	1·67	0·59	2·79	1·68
D. 0·40 per cent. carbon.	1·66	0·69	3·68	2·01
F. Mild steel 0·7 per cent. Mn.	0·97	0·39	2·32	1·23
Average	1·49	0·62	2·77	1·62
(4) *Colombo.*				
B. Mild steel, low Mn., high S. and P.	1·46	5·63	2·45	3·18
E. Medium carbon, low S. and P.	1·37	4·56	2·76	2·90
D. 0·40 per cent. carbon.	1·67	2·30	6·00	3·32
F. Mild steel 0·7 per cent. Mn.	4·56	5·30	3·15	4·34
Average	2·26	4·45	3·59	3·43
(5) *All four ports.*				
B. Mild steel, low Mn., high S. and P.	0·99	2·60	2·15	1·91
E. Medium carbon, low S. and P.	1·03	3·00	2·28	2·10
D. 0·40 per cent. carbon.	1·14	2·19	3·19	2·16
F. Mild steel 0·7 per cent. Mn.	1·53	2·60	2·58	2·19
Average	1·17	2·57	2·54	2·09

TABLE XVII.—CAST IRONS COMPARED WITH ROLLED IRONS.

General corrosion.

		Wastage: millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
R, Q. .	Cast irons.	0·006	0·029	0·301	0·111
N, P, M.	Rolled irons.	0·151	0·313	0·550	0·338
(2) *Auckland.*					
R, Q. .	Cast irons.	0·015	0·055	0·306	0·125
N, P, M.	Rolled irons.	0·349	0·230	0·409	0·329
(3) *Plymouth.*					
R, Q. .	Cast irons.	0·004	0·176	0·178	0·120
N, P, M.	Rolled irons.	0·929	0·482	0·486	0·632
(4) *Colombo.*					
R, Q. .	Cast irons.	0·306	1·243	0·564	0·705
N, P, M.	Rolled irons.	1·764	0·836	0·444	1·015
(5) *All four ports.*					
R, Q. .	Cast irons.	0·083	0·350	0·325	0·245
N, P, M.	Rolled irons.	0·797	0·465	0·472	0·579

TABLE XVIII.—THE CAST IRONS.

General corrosion.

		Wastage: millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
R. . .	Cold blast.	0·008	0·023	0·279	0·102
Q. . .	Hot blast.	0·005	0·035	0·324	0·121
Average		0·006	0·029	0·301	0·111
(2) *Auckland.*					
R. . .	Cold blast.	0·010	0·041	0·292	0·114
Q. . .	Hot blast.	0·020	0·070	0·321	0·137
Average		0·015	0·055	0·306	0·125
(3) *Plymouth.*					
R. . .	Cold blast.	0·008	0·149	0·174	0·110
Q. . .	Hot blast.	nil	0·204	0·182	0·129
Average		0·004	0·176	0·178	0·120
(4) *Colombo.*					
R. . .	Cold blast.	0·222	1·188	0·553	0·655
Q. . .	Hot blast.	0·390	1·299	0·575	0·755
Average		0·306	1·243	0·564	0·705
(5) *All four ports.*					
R. . .	Cold blast.	0·062	0·350	0·325	0·245
Q. . .	Hot blast.	0·104	0·402	0·350	0·285
Average		0·083	0·376	0·337	0·265

APPENDIX IV.

Table XIX.—The Copper Steels.

General corrosion.

	Wastage: millimetres.			
	Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*				
H. Mild steel with 2 per cent. copper.	0·050	0·208	0·450	0·236
G. Mild steel with ½ per cent. copper.	0·068	0·185	0·441	0·231
D. 0·40-per-cent. carbon steel.	0·105	0·414	0·464	0·328
(2) *Auckland.*				
H. Mild steel with 2 per cent. copper.	0·067	0·132	0·395	0·198
G. Mild steel with ½ per cent. copper.	0·101	0·081	0·407	0·196
D. 0·40-per-cent. carbon steel.	0·260	0·133	0·483	0·293
(3) *Plymouth.*				
H. Mild steel with 2 per cent. copper.	0·356	0·291	0·425	0·367
G. Mild steel with ½ per cent. copper.	0·628	0·371	0·411	0·480
D. 0·40-per-cent. carbon steel.	0·983	0·379	0·472	0·612
(4) *Colombo.*				
H. Mild steel with 2 per cent. copper.	1·541	1·211	0·504	1·086
G. Mild steel with ½ per cent. copper.	1·766	1·082	0·423	1·091
D. 0·40-per-cent. carbon steel.	1·850	1·063	0·581	1·163
(5) *All four ports.*				
H. Mild steel with 2 per cent. copper.	0·503	0·460	0·443	0·472
G. Mild steel with ½ per cent. copper.	0·641	0·430	0·420	0·499
D. 0·40-per-cent. carbon steel.	0·799	0·497	0·500	0·599

TABLE XX.—THE COPPER STEELS.

Pitting.

		Depth of deepest pit: millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions
(1) *Halifax.*					
H.	Mild steel with 2 per cent. copper.	nil	4·71	0·79	1·83
G.	Mild steel with ½ per cent. copper.	nil	4·18	2·25	2·14
D.	0·40-per-cent. carbon steel.	nil	4·72	0·96	1·89
(2) *Plymouth.*					
H.	Mild steel with 2 per cent. copper.	1·43	1·93	1·73	1·70
G.	Mild steel with ½ per cent. copper.	0·75	2·25	2·82	1·94
D.	0·40-per-cent. carbon steel.	1·09	1·04	2·14	1·42
(3) *Auckland.*					
H.	Mild steel with 2 per cent. copper.	nil	0·74	3·19	1·31
G.	Mild steel with ½ per cent. copper.	0·87	0·72	2·50	1·36
D.	0·40-per-cent. carbon steel.	1·66	0·69	3·68	2·01
(4) *Colombo.*					
H.	Mild steel with 2 per cent. copper.	1·31	4·41	3·33	3·02
G.	Mild steel with ½ per cent. copper.	1·77	5·09	3·01	3·29
D.	0·40-per-cent. carbon steel.	1·67	2·30	6·00	3·32
(5) *All four ports.*					
H.	Mild steel with 2 per cent. copper.	0·68	2·95	2·26	1·96
G.	Mild steel with ½ per cent. copper.	0·85	3·06	2·64	2·18
D.	0·40-per-cent. carbon steel.	1·14	2·19	3·19	2·16

TABLE XXI.—THE NICKEL STEELS.

General corrosion.

		Wastage : millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
L.	36 per cent. nickel.	0·003	0·050	0·341	0·131
K.	3½ per cent. ,,	0·029	0·062	0·430	0·174
D.	0·40-per-cent. carbon steel.	0·105	0·414	0·464	0·328
(2) *Auckland.*					
L.	36 per cent. nickel.	0·008	0·031	0·177	0·072
K.	3½ per cent. ,,	0·056	0·091	0·331	0·159
D.	0·40-per-cent. carbon steel.	0·260	0·133	0·483	0·293
(3) *Plymouth.*					
L.	36 per cent. nickel.	0·012	0·114	0·259	0·128
K.	3½ per cent. ,,	0·140	0·260	0·355	0·260
D.	0·40-per-cent. carbon steel.	0·983	0·379	0·472	0·612
(4) *Colombo.*					
L.	36 per cent. nickel.	0·012	0·213	0·149	0·124
K.	3½ per cent. ,,	1·501	0·835	0·386	0·908
D.	0·40-per-cent. carbon steel.	1·850	1·063	0·581	1·163
(5) *All four ports.*					
L.	36 per cent. nickel.	0·009	0·102	0·231	0·114
K.	3½ per cent. ,,	0·431	0·312	0·375	0·375
D.	0·40-per-cent. carbon steel.	0·799	0·497	0·500	0·599

TABLE XXII.—THE NICKEL STEELS.

Pitting.

		Depth of deepest pit: millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
L.	36 per cent. nickel.	nil	0·80	1·90	0·90
K.	3½ per cent. „	0·38	1·08	1·74	1·07
D.	0·40-per-cent. carbon steel.	nil	4·72	0·96	1·89
(2) *Plymouth.*					
L.	36 per cent. nickel.	nil	nil	1·28	0·43
K.	3½ per cent. „	0·54	1·62	3·23	1·80
D.	0·40-per-cent. carbon steel.	1·09	1·04	2·14	1·42
(3) *Auckland.*					
L.	36 per cent. nickel.	nil	nil	1·19	0·40
K.	3½ per cent. „	nil	0·95	2·71	1·22
D.	0·40-per-cent. carbon steel.	1·66	0·69	3·68	2·01
(4) *Colombo.*					
L.	36 per cent. nickel.	nil	1·73	1·38	1·04
K.	3½ per cent. „	1·19	perforated [1]	4·42	(5·87)
D.	0·40-per-cent. carbon steel.	1·67	2·30	6·00	3·32
(5) *All four ports.*					
L.	36 per cent. nickel.	nil	0·63	1·44	0·69
K.	3½ per cent. „	0·53	(3·91)	3·02	(2·82)
D.	0·40-per-cent. carbon steel.	1·14	2·19	3·19	2·16

[1] Perforation probably due to mollusc action. For the averages the depth of the pitting is tentatively taken as 12 mm. (about ½ inch).

TABLE XXIII.—CHROMIUM STEEL.

General corrosion.

		Wastage : millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
J.	13-per-cent. chromium steel.	0·012	0·064	0·277	0·117
D.	0·40-per-cent. carbon steel.	0·105	0·414	0·464	0·328
(2) *Auckland.*					
J.	13-per-cent. chromium steel.	0·019	0·064	0·269	0·117
D.	0·40-per-cent. carbon steel.	0·260	0·133	0·483	0·293
(3) *Plymouth.*					
J.	13-per-cent. chromium steel.	0·021	0·144	0·157	0·110
D.	0·40-per-cent. carbon steel.	0·983	0·379	0·472	0·612
(4) *Colombo.*					
J.	13-per-cent. chromium steel.	0·158	0·407	0·288	0·284
D.	0·40-per-cent. carbon steel.	1·850	1·063	0·581	1·163
(5) *All four ports.*					
J.	13-per-cent. chromium steel.	0·052	0·170	0·248	0·157
D.	0·40-per-cent. carbon steel.	0·799	0·497	0·500	0·599

TABLE XXIV.—THE CHROMIUM STEELS.

Pitting.

		Depth of deepest pit: millimetres.			
		Aerial.	Half-tide.	Total immersion.	All three conditions.
(1) *Halifax.*					
J.	13-per-cent. chromium steel.	nil	1·18	3·20	1·46
D.	0·40-per-cent. carbon steel.	nil	4·72	0·96	1·89
(2) *Plymouth.*					
J.	13-per-cent. chromium steel.	nil	0·67	0·83	0·50
D.	0·40-per-cent. carbon steel.	1·09	1·04	2·14	1·42
(3) *Auckland.*					
J.	13-per-cent. chromium steel.	nil	0·63	1·40	0·68
D.	0·40-per-cent. carbon steel.	1·66	0·69	3·68	2·01
(4) *Colombo.*					
J.	13-per-cent. chromium steel.	nil	6·50	6·50	4·33
D.	0·40-per-cent. carbon steel.	1·67	2·30	6·00	3·32
(5) *All four ports.*					
J.	13-per-cent. chromium steel.	nil	2·24	2·98	1·74
D.	0·40-per-cent. carbon steel.	1·14	2·19	3·19	2·16

APPENDIX V.

TABLE XXV.—PARTICULARS OF SPECIMENS TESTED BY EXPOSURE TO RIVER WATER.

Mark.	Material.	Analysis: per cent. C.	Si.	S.	P.	Mn.	Cr.	Fe (by difference).	Heat-treatment.	Condition of surface.	Brinell hardness numbers.	Corresponding mark in the Sea-action Committee's research.
A.1	Chromium steel, hard grade.								As forged.	With scale.	550	J.
A.2	,,								800° C., furnace.	,,	212	—
A.3	,,	0·36	0·22	0·030	0·020	0·13	13·57	85·67	,,	Pickled.	218	—
A.4	,,								800° C., furnace. 920° C., oil. 250° C., air.	Polished.	477	—
B.1	Chromium steel, soft grade, with 0·5 per cent. Si.								As rolled.	With scale.	367	S.
B.2	,,								800° C., furnace.	,,	178	T.
B.3	,,	0·12	0·53	0·042	0·015	0·21	11·90	87·18	,,	Pickled.	168	U.
B.4	,,								930° C., oil. 600° C., water.	With scale.	248	W.
B.5	,,								930° C., oil. 600° C., water.	Pickled.	252	—
C.1	Chromium steel, soft grade, with 2·4 per cent. Si.	0·14	2·40	0·030	0·020	0·26	11·97	85·18	800° C., furnace.	Pickled.	215	—

TABLE XXVI.—CORROSION OF CHROMIUM STEEL (SOFT GRADE) BY RIVER-WATER, WHEN IN CONTACT WITH MILD STEEL OR CAST IRON.

The chromium steel (specimen B.3) used in these tests was annealed by slow cooling from 800° C. and the scale was subsequently removed by pickling.

Period of exposure—566 days.

Material exposed.	Loss in weight when exposed as indicated: milligrams.			
	Alone	In contact with:		
		Non-rusting chromium steel.	Mild steel.	Cast iron.
Chromium steel, soft grade.	181		(*a*) 35 *With both mild steel and cast iron:* (*d*) 11.	(*b*) 10
Mild steel	853	(*a*) 1363 *With both chromium steel and cast iron:* (*d*) 860		(*c*) 782
Cast iron	961 *	(*b*) 1,456 *With both chromium steel and mild steel:* (*d*) 1,082	(*c*) 829	

Similar letters attached to the figures indicate that they belong to the same test, the separate figures showing the respective losses of each of the separate materials.

* 555 days only.

TABLE XXVII.—PARTICULARS OF SPECIMENS EXPOSED TO ELECTROLYTIC CORROSION AT HECLA WORKS, SHEFFIELD.

Material.	Analysis: per cent.							Heat-treatment.
	C.	Si.	S.	P.	Mn.	Cr.	Fe (by difference).	
Chromium steel, soft grade (1)	0·12	0·53	0·042	0·015	0·21	11·90	87·183	Heated to 800° C. and cooled slowly.
Chromium steel, soft grade (2)	0·15	1·20	0·021	0·010	0·24	11·24	87·139	
Mild steel	0·21	0·19	0·089	0·066	0·66	—	98·785	As forged.
Cast iron	2·76 (*a*) 0·49 (*b*) 3·25	1·99	0·106	0·61	0·52	—	93·524	As cast.

(*a*) Graphitic carbon. (*b*) Combined carbon.

TABLE XXVIII.—CONDITION OF PARTLY MACHINED SPECIMENS OF CHROMIUM STEEL AFTER EXPOSURE TO RIVER WATER.

Mark.	Type of material.	Heat-treatment.	Condition of surface.	Interim examination after 5½ months.		Final examination after 20 months.[1]
				Portion not machined.	Machined portion.	Machined portion.
A.2	Chromium steel, hard grade.	Annealed.	With scale.	Slightly rusted.	Stained.	Good, slight pitting.
A.3	,,	,,	Pickled.	Brown patches, slight pitting.	Brown patches, slight pitting.	,,
B.1	Chromium steel, soft grade, with 0·5 per cent. Si.	As rolled.	With scale.	Pitted and rusted.	Badly pitted.	(Not further tested.)
B.2	,,	Annealed.	,,	In good condition.	Tarnished.	Good, slight pitting.
B.4	,,	Quenched and tempered.	,,	Slightly pitted and rusted.	Slightly pitted and rusted.	,,
B.5	,,	Quenched and tempered.	Pickled.	Slightly rusted, surface good.	Slightly rusted, surface good.	Pitted slightly more than B.4.
C.1	Chromium steel, soft grade, with 2·4 per cent. Si.	Annealed.	Pickled.	Somewhat rusted.	Very good.	Local pitting.

[1] All the specimens were covered with a thin layer of salts from the water.

TABLE XXIX.—CONDITION OF PLAIN SPECIMENS OF CHROMIUM STEEL AFTER EXPOSURE TO RIVER-WATER.

The specimens, excepting B.4 and B.5, were exposed under conditions of intermittent immersion over a total period of 23 months, actual exposure occupying 553 days.
Specimens B.4 and B.5 commenced their test rather later, total exposure occupying in their cases 535 days.
The surface of each of the specimens has a different appearance along two adjacent edges from the rest of the surface, and the amount and character of the corrosion is also in most cases different (*Fig. 10*, p. 51). This is where they have been in contact with the wooden frame in which they rested.

Mark.	Material.	Rolled or forged.	Heat-treatment.	Original condition of surface.	Visual examination. Where in contact with supporting frame.	Visual examination. Other portions of surface.	Visual examination. Ends.
					GROUP 1.—Chromium steel—Hard grade. Analysis: 0·36 C.; 0·22 Si.; 0·13 Mn.; 13·57 Cr.		
A.1	Chromium steel, hard grade.	Forged.	As forged.	With scale.	[1] Scale off in small patches with areas of rusting. Only very slight traces of rust on opposite edges where specimen appears to also have been in frame. Corner where previously polished for Brinell test, apart from very small areas of rust, still quite bright.	Small areas of rusting, but otherwise practically unaffected.	Slight pitting at end "A." Other end practically unaffected.
A.2	,,	,,	Annealed.	,,	Slight pitting, areas of rusting much less pronounced than on other parts of surface.	Local pitting and scale off in small areas, mostly within ¼ inch from one edge. Practically the whole of both surfaces covered with rust, heavy in places.	Scale off in patches at end "A," slight pitting on opposite end.
A.3	,,	,,	,,	Pickled.	Local pitting and rust adhering in places.	Patches of rust, otherwise practically unaffected.	End "A," rust adhering in places, slight pitting on opposite end.
A.4	,,	,,	Quenched and tempered.	Polished.	Apparently corroded in large grey and red patches on surfaces and edges, but these seem to be merely deposits and will probably clean off, leaving whole surface bright, except on end "A," where the surface is gone in small patches, and traces of pitting generally with one deep pinhole. No marked difference in this case between portions in frame and other parts. This specimen, owing to its preserving its original polish, is in marked contrast with all the others.		An area of local pitting at one end.
					GROUP 2.—Chromium steel—Soft grade. {B.—0·50 per cent. Si.; 0·12 C.; 0·53 Si.; 0·042 S.; 0·015 P.; 0·21 Mn.; 11·90 Cr. C.—2·40 per cent. Si.; 0·14 C.; 2·40 Si.; — — 0·26 Mn.; 11·97 Cr.		
B.1	Chromium steel, soft grade (0·5 per cent. Si).	Rolled.	As rolled.	With scale.	[1] Very badly pitted, patches of rust. Other adjacent edges of surface badly pitted to about the same extent but not rusted. Scale off in places.	Scale off in places on centre portion of surfaces, very slight pitting and specks of rust. Corner previously polished and ball tested has lost most of its bright surface.	Both ends badly pitted.
B.2	,,	,,	Annealed.	,,	Local pitting rather bad. Less rusted than other portions.	Scale almost intact. Areas of rusting on both surfaces, rather bad in places.	Both ends badly pitted.
B.3	,,	,,	,,	Pickled.	[1] Local deep pitting with areas of rusting. Machined edges very badly pitted. Other long side not so badly pitted, only traces of rust but darker appearance than other parts of specimen.	Parts slightly corroded but in a fairly uniform manner by small pits still leaving rather large areas of surface unattacked.	End A attacked at one corner, other end pitted rather badly.
B.4	,,	,,	Quenched and tempered.	With scale.	Rather bad local pitting. Other parts slightly rusted.	Rather bad local pitting, scale off in places. Lightly rusted with heavy specks on practically the whole of one surface, specks of rust on the other.	Both ends badly pitted.
B.5	,,	,,	,,	Pickled.	Badly pitted.	Patches of small areas of corrosion, rust adhering in places, some rather heavy specks. A corner previously polished for ball test now covered apparently with dark deposit and specks of rust.	Both ends badly pitted.
C.1	Chromium steel, soft grade (2·4 per cent. Si).	,,	Annealed.	,,	Very badly pitted (confined to this area) small patches of rust in places, but more free than other parts of specimen.	Much corroded in a fairly uniform manner by small pits, still leaving a good deal of the original surface unattacked. Slightly rusted nearly all over and heavy in places.	Rusted and badly pitted.

[1] In some cases the specimens when taken out at an intermediate period for inspection were replaced with the opposite edge in contact with the frame, so that both of the long edges show similar characters.

TABLE XXX.—CORROSION OF CHROMIUM STEEL (SOFT GRADE) BY (1) SHEFFIELD TAP-WATER, (2) ARTIFICIAL SEA-WATER, WHEN IN CONTACT WITH MILD STEEL OR CAST IRON.

Period of exposure: 14 days.

Material exposed.	Loss in weight, when exposed as indicated: milligrams.				Corrosive medium.
	Alone.	In contact with: Chromium steel.	In contact with: Mild steel.	In contact with: Cast iron.	
Chromium steel, soft grade.	1·0		(a) nil	(b) 0·5	Tap-water.
	23·0		(d) nil	(e) nil	Sea-water.
Mild steel	63·5	(a) 74·5		(c) 42·0	Tap-water.
	75·5	(d) 115·5		(f) 24·0	Sea-water.
Cast iron	111·0	(b) 71·0	(c) 56·5		Tap-water.
	132·0	(e) 137·0	(f) 125·5		Sea-water.

Similar letters attached to the figures indicate that they belong to the same test, the separate figures showing the respective losses of each of the separate materials.

APPENDIX VI.

TABLE XXXI.—PARTICULARS OF SPECIMENS TESTED BY EXPOSURE ON MARINE BUOYS

Mark.	Material.	Analysis: per cent.							Heat-treatment.	Condition of surface.	Brinell hardness number.
		C.	Si.	S.	P.	Mn.	Cr.	Fe (by difference).			
M.S.	Mild steel.	0·15	0·010	0·042	0·008	0·49	—	99·30	As rolled.	Without scale.	120
G.	Chromium steel, hard grade.								650° C., air.	With scale.	302
	,,	0·34	0·21	0·015	0·017	0·18	14·06	85·18	,,	Without scale.	302
	,,								775° C., furnace.	With scale.	238
—	Mild-steel rivets.	0·21	0·07	0·032	0·032	0·48	—	—	—	—	—

TABLE XXXII.—RESULTS OF CORROSION-TESTS BY EXPOSURE ON TRINITY HOUSE BUOYS.

The analyses of the materials are shown in Table XXXI.

Name of buoy.	Location.	Commencement of test.	Date of examination.	Period of exposure: Months.	Condition of specimens.	
					Above-water specimens.	Under-water specimens.
					(a) Reports of visual examinations.	
North Brake.	Off Ramsgate. (Open-sea conditions.)	17 Nov., 1922.	5 July, 1923.	7½	All chromium-steel specimens only slightly rusted. Mild steel badly rusted. Rust can be flaked off, showing pits underneath. All rivets have a scale of rust.	Slight coating of weed which is easily rubbed off. Chromium steel specimens have a dark deposit but no rust. Mild steel rusted, but not badly. Corrosion in general less than for above-water specimens. All rivets have a scale of rust.
			14 Dec., 1923.	13	Practically no difference.	Heads of the two rivets at the end of chromium-steel specimen G.6 have disappeared. Stamped numbers on mild-steel specimen M.S.6 almost obliterated by rust.
			23 Dec., 1923.	13	Buoy removed.	Buoy removed.
Nore Sand.	Thames estuary. (Estuarine conditions.)	15 Nov., 1922.	16 Feb., 1924.	15	Chromium-steel specimen slightly rusted, but surface smooth and bright under the rust. Mild steel badly rusted; number mark obliterated. Rivet heads partly rusted away.	Chromium steel not rusted. White deposit probably from the attached shellfish. Surface of mild steel eaten away and rough. Rivet-heads partly rusted away, and disappeared altogether from chromium steel specimen G.10.
			19 Feb., 1924.	15	Buoy removed.	Buoy removed.

North Cliffe.	Harwich harbour. (Harbour conditions.)	30 Nov., 1922.	5 July, 1923.	7	Chromium-steel specimens discoloured but bright under discoloration. Mild steel slightly attacked.	Marine growth on all specimens. Chromium-steel specimens bright. Mild steel slightly deteriorated.
			15 Dec., 1923.	12½	Chromium - steel specimens slightly rusted or discoloured but bright underneath coating. Mild steel has rusted further.	Practically as at previous examination.
			19 June, 1924.	18½	Chromium-steel specimens G.1 and G.13 pitted, but G.7 smooth. All slightly rusted. Mild steel badly rusted and much pitted.	Chromium - steel specimens unattacked. Mild steel badly rusted and much pitted. All rivet-heads badly eaten away. Mild-steel specimen hanging by one rivet.
			19 June, 1924.	18½	Buoy removed.	Buoy removed.

(b) *Loss in weight on completion of exposure.*

Name of buoy.	Location.	Period of exposure: months.		Losses in weight: grams.			
				Mild steel. As rolled. Scale removed.	Chromium steel.		
					Cooled in air from 650° C.		Annealed at 775° C.
					With scale.	Without scale.	Without scale.
North Brake.	Off Ramsgate. (Open-sea conditions.)	13	Above water.	76	3	nil	nil
			Below water.	148	3	+1	nil
Nore Sand.	Thames estuary. (Estuarine conditions.)	15	Above water.	97	9	6	7
			Below water.	72	2	nil	2
North Cliffe.	Harwich harbour. (Harbour conditions.)	18½	Above water.	97	6	7	2
			Below water.	156	4	nil	+1

APPENDIX VII.

TABLE XXXIII.—RESULTS OF EXPOSURE TESTS ON STEELS SELECTED FROM THE SEA-ACTION COMMITTEE'S RESEARCH IN THE ATMOSPHERE AT BIRMINGHAM.

Period of test: 6 years.

Specimens 3 inches by ½ inch by 12 inches, exposed with their scale on.
All the materials in the untreated condition.
The analyses of the materials are shown in *Fig. 11* (p. 59).

Distinguishing mark.	Type of steel.	After exposure at Birmingham.		Losses in the Sea-Action Committee's Research: grams per 1,000 square centimetres.			
				Aerial exposure for 5 years.			
		Appearance.	Loss in weight: grams per 1,000 square centimetres.*	Halifax.	Auckland.	Plymouth.	Colombo.
		Carbon steels. (Section II of Sea-Action Committee's research.)					
B.	Mild steel, low Mn. and high S. and P.	Very rough. Cellular depressions of greater area than less attacked network.	779	137	262	854	1,441
F.	Mild steel with 0·7 per cent. Mn.	"	549	103	150	784	1,496
E.	Medium carbon steel with low S. and P.	"	485	84	172	630	1,560
D.	0·40-per-cent. carbon steel.	"	590	82	203	766	1,444

		Special steels. (Section III of Sea-Action Committee's research.)					
G.	Mild steel with ½ per per cent. copper.	Rough. Cellular depressions of greater area than less attacked network.	321	53	79	490	1,379
H.	Mild steel with 2 per cent. copper.	Very rough. Longitudinal median area rather less attacked.	337·8	39	52	278	1,204
K.	3½-per-cent. nickel steel.	Very rough. Uniformly mottled like "B."	200	23	44	109	1,172
J.	Chromium steel.	Generally smooth, but several patches, mostly elongated in direction of rolling (varying in size up to about ½ inch long), show fairly heavy attack.	7·19	9	15	16·6	123
L.	36-per-cent. nickel steel.	Very smooth. Still some small particles of scale adhering.	27·6	2·3	6	9·1	9

* Only the portion (namely, 9¾ inches in length) not embedded in the wooden supports is considered as the area exposed.

APPENDIX VIII.

TABLE XXXIV.—PARTICULARS OF SPECIMENS IMMERSED IN THE GULF OF PARIA.

Mark.	Analysis: per cent. C.	Si.	S.	P.	Mn.	Cr.	Ni.	W.	Cu.	Treatment.	Condition of surface.	Used in series numbers.
Mild steel.												
3,042	0·09	0·03	0·026	0·014	0·53	—	—	—	—	As rolled.	With rolling scale.	1
3,042	0·11	0·11	0·018	0·010	0·47	—	—	—	—	”	”	2
Mild steel containing copper.												
3,714	0·17	0·10	0·023	0·027	0·43	—	—	—	0·39	As forged.	With forging scale.	1
3,714	0·11	0·13	0·025	0·012	0·37	—	—	—	0·29	850° C., air.	”	2
3,714	0·11	0·13	0·025	0·012	0·37	—	—	—	0·29	”	Sand-blasted.	2
Nickel steels.												
3,450	0·09	0·09	0·040*	0·020*	0·77	—	36·45	—	—	As rolled.	With rolling scale.	1
3,450	0·12	0·09	0·040*	0·020*	0·87	—	36·5	—	—	”	”	2
3,450	0·08	0·07	0·042	0·019	1·00	—	36·4	—	—	As rolled and welded.	Pickled.	2
3,611	0·31	0·18	0·029	0·038	0·54	—	3·75	—	—	As forged.	With forging scale.	2

29-per-cent. chromium steel.													
4,344	0·69	0·38	0·070*	0·040*	0·30	29·2	—	—	—	700° C., air.	Pickled.	1	
Nickel-copper steels.													
3,236	0·35	0·28	0·046	0·024	1·06	—	1·71	—	0·77	As forged.	With forging scale.		2
1,251A	0·24	0·37	0·100*	0·060*	1·37	—	2·67	—	1·38	,,	,,		2
1,251B	0·20	0·35	0·100*	0·060*	1·20	—	2·86	—	2·96	,,	,,		2
Nickel-chromium steels (and with additions of other elements).													
3,765	0·06	0·14	0·040*	0·030*	0·52	10·1	19·8	1·94	1·81	950° C., air.	With rolling scale.		2
3,765	0·06	0·14	0·040*	0·030*	0·52	10·1	19·8	1·94	1·81	,,	Pickled.		2
4,375	0·14	0·45	0·060	0·035	0·49	10·0	19·6	2·00	2·10	,,	,,	1	
3,754	0·12	0·43	0·020*	0·020*	0·24	18·8	8·10	—	—	1,100° C., air.	,,	1	
4,310	0·35	1·50	0·045*	0·025*	0·50*	20·0	7·00	4·00	—	950° C., air.	With rolling scale.		2
4,310	0·35	1·50	0·045*	0·025*	0·50*	20·0	7·00	4·00	—	,,	Pickled.		2
3,760	0·13	0·30	0·040*	0·030*	0·39	17·0	2·10	—	—	1,000° C., air. 600° C., air.	With forging scale.		2
3,760	0·13	0·30	0·040*	0·030*	0·39	17·0	2·10	—	—	,,	Sand-blasted and pickled.		2

* Estimated figures.

TABLE XXXV.—RESULTS OF EXPOSURE TESTS ON VARIOUS SPECIAL STEELS BY TOTAL IMMERSION IN THE GULF OF PARIA.

First Series.

Total period of immersion: 139 days.

(After the first 63 days the specimens were cleared of barnacles and the corrosion products partially removed. They were then returned to the sea for the remaining period of 76 days.)

Mark.	Type of steel.	Loss in weight: grams per 1,000 square centimetres.	Special features.	
3,450	36 per cent. nickel.	33·0	Absence of pitting. Large proportion of both faces bears original scale. Undoubtedly the best of the series.	
3,754	18 per cent. Cr, 8 per cent. Ni.	11·09	Bad pitting on both faces. Especially bad on edges.	Apart from pitting, surface remains bright.
4,375	10 per cent. Cr, 20 per cent. Ni, 2 per cent. W, 2 per cent. Si.	31·1	Pitting on faces localized to small areas. Deep on front face, but only pin-holing on back. Edges roughened.	
4,344	29 per cent. chromium.	37·8	Most severely pitted of all. Small pits distributed over surface. Deep pits near and at edges.	
3,714	Mild steel with 0·39 per cent. copper.	64·8	Practically no pitting. General corrosion more severe towards edges. Distinctly smoother than mild steel.	
	For comparison:—			
3,042	Mild steel.	58·9 *	Very rough, with pitting on both faces. Some of original scale remains, more especially on the back surface.	

* This figure is low owing to incomplete removal of products of corrosion.

Table XXXVI.—Results of Exposure Tests on Various Special Steels by Total Immersion in the Gulf of Paria.

Second series.

Period of immersion: 6 months.

Mark on Specimen.	Type of steel.	With original scale. Loss in weight: grams per 1,000 square centimetres.	With original scale. Special features.	Surface cleaned. Surface prepared by:	Surface cleaned. Loss in weight: grams per 1,000 square centimetres.	Surface cleaned. Special features.
Nickel steels.						
3,611	3¾ per cent. nickel.	90·0	Selectively attacked at centre of front surface. Scale still adheres to most of remaining surfaces. Edges slightly rounded.	—	—	—
3,450	36 per cent. nickel.	48·2	Scale detached in small areas distributed over both surfaces. Line of small pits along edges at back.	—	—	—
3,450	36 per cent. nickel, welded.	—	—	Ground and pickled.	49·0	Slight selective attack. General corrosion rather deeper along line of contact with frame, and near, but not at, the weld.
Nickel-chromium steels.						
3,765	10 per cent. Cr, 20 per cent. Ni, 2 per cent. W, 2 per cent. Cu.	33·7	Deep pitting on edges. Some scale remaining on faces, but rest smooth.	Pickled.	3·1	Pattern apparently formed by mollusc action. Slight pitting on back surface; pitting and one crater on edge.
4,310	20 per cent. Cr, 7 per cent. Ni, 4 per cent. W, 1½ per cent. Si.	12·3	Deep selective attack; almost perforated. Some scale remains. Edges deeply pitted.	Pickled.	30·7	Scattered localized attack on both faces, and perforated at edges. Little general corrosion.
3,760	17 per cent. Cr, 2 per cent. Ni.	27·6	Deep selective attack, with one deep crater ⅜-inch diameter. Some scale remains. No pitting on edges.	Pickled.	46·0	Very severe deep selective attack on faces and edges. Shallow oval crater 1 inch by ½ inch near edge. Bright where not attacked.
Copper steel.						
3,714	0·29 per cent. copper.	98·0	Numerous craters, specially deep on front surface. Edges irregular.	Sand-blasted.	Not recorded.	Much roughened on front surface, but smooth at back and on edge. Practically no pitting.
Nickel-copper steels.						
3,236	1¾ per cent. Ni, ¾ per cent. Cu.	84·0	Some scale remains. Tendency to pitting on front face and rather deeper attack on line of contact with frame.	—	—	—
1,251A.	2¾ per cent. Ni, 1¼ per cent. Cu.	78·0	Selective attack, heavy in centre of front face. Some scale remains. Rather deeper attack on line of contact with frame.	—	—	—
1,251B.	2¾ per cent. Ni, 3 per cent. Cu.	100·0	Large areas deeply cratered. Edges heavily rusted and rounded.	—	—	—
Mild steel (For comparison).						
3,042	0·11 per cent. carbon.	117·0	Original scale remains over most of both surfaces. Attack fairly uniform. Few small craters on front face. Edges thinned.	—	—	—

Discussion.

Sir Robert Hadfield.

Sir ROBERT HADFIELD said that the Paper had been prepared some considerable time ago, and in the period which had since elapsed some progress had occurred which the Authors would have liked to have taken into account, but that had not been possible. He would like to say how indebted he had been for the very kind help and encouragement given to him, during his 20 years' membership of the Sea-Action Committee. He would also like to express his indebtedness to his colleague, Mr. S. A. Main, who had rendered him yeoman service with the preparation of the present Paper, and to the many willing helpers in his firm's research department in Sheffield. Next, he would refer especially to the late Sir John Wolfe Barry, Past-President Inst. C.E.; one of the things Sir John had wanted to know was whether the 12- to 14-per-cent. chromium steel could safely be used for cutlery. Sir John had heard that owing to the formation of certain oxides the steel was liable to be poisonous. Sir Robert Hadfield had given him the assurance that, so far as was then known, he had no need to fear, and had, at Sir John's request, written him a letter to that effect. The best answer on that point was that millions of cutlery-articles made from chromium steel were now in daily use throughout the world, and great credit was due to the manufacturers of those special steels. He would also like to refer to the Department of Scientific and Industrial Research; the sea-water corrosion-research could not have been initiated and carried on but for the timely assistance given by the Department, which had given £10,600. A further sum of £11,700 had been given privately, making a grand total of £22,300. Those donations had materially assisted the work, and most important additions to the knowledge of the subject had been made. He was confident that the research would be of the greatest use to the world. He did not wish to compare the work of the Sea-Action Committee with that of other Committees, which were also doing splendid work, but it had done work in certain directions not included in other researches.

One of the first of the various ferrous materials to be examined by the Committee was wrought iron, one of the commercial materials approaching most nearly to pure iron. He was able that evening to exhibit what was probably the most remarkable specimen of wrought iron in the world, from that wonderful example of the work of the metallurgist of about A.D. 300, the Delhi Iron Pillar; that pillar dated from some 1,600 years before the introduction of the modern forge, steam hammer or hydraulic press. There were all

Sir Robert Hadfield.

kinds of strange things connected with that pillar, and many which were not understood. In view of the early date at which it was produced, it was especially remarkable on account of its extraordinary size and weight, being 23 feet 8 inches in length and about 6 tons in weight. The special point, however, was not its size and weight, for there were still larger specimens in India dating from a later period, but that everyone who had in any way examined or dealt with it had been baffled to know how a pillar of such considerable size and weight had been manufactured. It had gained the admiration of the world by its beauty of form and by its extraordinary resistance to corrosion, the latter being probably largely due to the dryness and cleanliness of the atmosphere in which it was located. The legend that its composition was of some special nature which would resist corrosion was not correct, because it was, in fact, made of what was known to-day as wrought iron. In 1912 he was fortunate enough to be entrusted with about 2 ounces of material taken from the pillar, through the kind help of Sir John Marshall, then Director of Archæology in India. From that specimen he had been able to obtain interesting analytical and other data, including chemical composition, hardness and microstructure. The actual composition of the pillar was shown to be :—

Carbon, 0·080 per cent.; silicon, 0·046 per cent.; sulphur, 0·006 per cent.; phosphorus, 0·114 per cent.; manganese, nil; iron, 99·72 per cent.; total, 99·966 per cent.

The Brinell ball hardness-number was 188, and the specific gravity 7·81. Although this enabled the material of which the pillar was composed to be identified, the method of manufacture of the pillar was still unknown. Its largest diameter was 16½ inches, tapering to the smallest diameter of about 12½ inches at a height of 22 feet above the ground. The bulbous underground portion of the pillar was no less than 28 inches in diameter by 20 inches in depth, and was anchored to the ground by crossbars which were also of wrought iron.

The specimens from the pillar sent to him were tested in the ordinary way for resistance to corrosion, and a small sample weighing a few grams was left in the laboratory atmosphere overnight; the next morning it was found covered with red rust, as any other wrought-iron specimen would have been. He could definitely state that the wrought iron of which the pillar was made had no qualities different from those possessed by ordinary wrought iron, such as the modern specimens used in the research dealt with in the Paper. There was thus obtained an exact comparison between the wrought iron of to-day and that made 1,600 years ago. There was another point of particular interest which he had elucidated. Included amongst,

and attached to, the specimens received from Sir John Marshall were scales taken from the surface of the pillar at its base, and described as "rust-scales." Whilst not wishing to throw any doubt on that description—the scales in question were undoubtedly from the pillar—it was not clear whether they were produced by the heat-treatment of the pillar for forging-purposes or were actual rust from contact with the ground, which was occasionally damp. The present belief was that they were heat-treatment scales. At the present time an X-ray examination was proceeding to ascertain their true nature. There were several other extraordinary facts which were being discovered with regard to the pillar, and it was hoped that they would shortly be embodied in a Paper on the subject. Sir Robert Hadfield.

Sir Robert Hadfield then showed on the epidiascope (for the loan of which he expressed his gratitude to Sir William Bragg, and for the operation of which he thanked Mr. W. J. Green, Sir William's assistant), six specimens, including three from the Delhi Iron Pillar.

It might be asked what was the use of the research, and he would therefore like to give a practical instance of its utility. About 10 or 12 years ago, when the research had been in progress for some time, he decided to try to use in a practical way the information which had been gained, and a large number of different kinds of iron and steel sheets were made into roofing. The atmosphere at Sheffield contained a great deal of smoke, some of which was sulphurous. His firm had something like 60 acres of buildings, and consequently they found the cost of replacing roof-sheeting to be considerable. They put down four special specimens, including some very pure iron and steel and also steel containing 0·35 per cent. of copper. The tests had not yet been completely finished, but it would appear that the copper steel was easily the best; he did not claim that as a novelty, because in America a large number of tests had been carried out, and no doubt such tests had been made elsewhere in England, but roofing sheets of that composition would, he believed, prove highly beneficial to his firm. When further buildings were erected, as was anticipated, a large number of sheets would be required, and he thought that steel containing 0·35 per cent. of copper would undoubtedly be specified. He did not infer that that copper steel would be of use in resisting corrosion due to sea-water, but the Sea-Action Committee's tests included an aerial test and the above results therefore came within the scope of the research. He mentioned that fact to show that, as a result of the research work which had been done, his own firm had benefited, as could any other firm which cared to look into the matter, by saving money in the cost of roofing-sheets by using a 0·35-per-cent. copper steel.

Mr. S. A. Main remarked that, for the most complete appreciation Mr. Main.

Mr. Main.

of the work which had been done by the Committee of The Institution, it was necessary to refer to the reports of the Committee; only in that way could the details be studied. It would be found in the Paper that the bars which had had their scale removed before exposure were referred to as "scaled bars," which was the ordinary term used in such circumstances by the engineer; in metallurgical circles, however, it was customary to refer to such bars as "descaled." If that point were appreciated, no confusion should arise in the discussion.

He would also like to say that his examination of the results had impressed him very much with the efficiency of the research as a whole. Each specimen had been chosen to answer a particular point, and every one of those points was something that the engineer wanted to know; for instance, the influence of climate and method of exposure, and the effect of rivets and bolts, had all been investigated. He thought it could be said that no single bar in the research had failed to yield the result that was expected of it. That was a very great thing to say of such a big research, and it might be expected to be true for the remaining bars, which had yet to be taken out after 15 years' exposure.

Mr. Wilson.

Mr. M. F-G. Wilson remarked that he was especially interested in the Paper, as it dealt with a section of the investigations of the Sea-Action Committee (of which he was an original member, and of which for some years he had been Chairman), that had been carried on for many years, and it also gave him an opportunity, which he was very glad to take, of thanking Sir Robert Hadfield for the very great trouble he had taken to assist the investigations of the Committee. Sir Robert had advised the first Chairman, the late Sir William Matthews, Past-President Inst. C.E., on the general scheme of the tests and as to how they should be carried out, and Mr. Wilson thought it would be agreed that the tests had been arranged so as to overlap and check each other in a very satisfactory manner. Taking, for example, any one bar, say the wrought-iron "N," and one station, say Plymouth, three bars were exposed above high water-level for 5 years, 10 years, and 15 years; at half-tide level three more such bars were exposed, and three more were placed below low water-level, making in all nine "N" bars at Plymouth. The whole of the experiments were repeated in three situations abroad, the same method of exposure being followed. A similar set was sent to Colombo, where the conditions were tropical and very moist; in fact, the half-tide specimens might almost be said to be under water, seeing that the tide rose only about 2 feet. Another set was sent to Auckland, in New Zealand, where the conditions were semi-tropical, and another set to Halifax, Nova Scotia. It would be seen that the experiments overlapped each other, and enabled

a very comprehensive set of results to be obtained. In addition, a further set of fresh-water experiments was carried out at Plymouth. Mr. Wilson.

In regard to the half-tide and complete immersion experiments, he had tried to determine by comparative figures an order of merit for the metals with regard to their resistance to corrosion, and he had adopted the method used by Dr. Newton Friend in his report. Sir Robert Hadfield had taken the actual thickness of the corrosion in decimals of a millimetre, but Mr. Wilson thought that Dr. Newton Friend had adopted a somewhat simpler method; he actually weighed each bar before and after exposure, and called the difference in weight the amount of corrosion; that was, the corrosive loss. Having obtained that for the whole set of experiments in one place, Dr. Friend took the wrought-iron bar "N" and called the value of its loss 100, and worked out all the rest accordingly, so that all the losses came out as a percentage of that of the standard bar. Mr. Wilson thought that those figures might be of practical use to engineers; they were set out in Table XXXVII (p. 106).

The best steel of all, as would have been gathered from the Paper, was the 36-per cent. nickel steel "L," and, again, taking the figure for wrought iron as 100, the figure for that steel was 36. The cost, however, of that steel would probably be very high. He did not know whether the Authors would give in their reply the comparative costs for that nickel steel as compared with the other steels. If, for purposes of economy, the nickel steel were used only in special positions, it would necessarily be in contact with other steels, and thus give rise to the electrolytic action to which the Authors had referred. The result would then be that the other steel would corrode away very much more quickly than normally, and that fact would have to be put against the value of the nickel steel. The next steel in order of merit was the 13·57-per-cent. chromium steel "J," with the relatively high figure of 51. That, so far as general corrosion was concerned, was very satisfactory, and it was exceedingly good in the air; in the water, however, it corroded badly, and very severe pitting took place; many of the bars were completely perforated, so that the use of that steel could not be considered for the purpose which the Committee had in mind. Next came the 3½-per-cent. nickel steel, whose comparative value was 72. That steel was very satisfactory in the air, and it had some advantages in the water, but it was not entirely reliable, seeing that there were several cases where bad corrosion had occurred, although on the whole it might be looked upon as being fairly satisfactory. The best steel for general purposes appeared to be the copper steel to which Sir Robert Hadfield had referred. The average figure for the 0·5-per-cent. copper steel was 90, which showed an appreciable advantage

Mr. Wilson. over the rolled iron, whose figure was 112. Generally speaking, Mr. Wilson thought that that would be the most useful steel for engineers to consider for use in sea-water, and he presumed it would not entail much extra cost. Another interesting fact which emerged was that, taking the rolled irons "M," "N," and "P," their average loss came out at 112, while the rest of the steels "A," "C," "D," "E," and "F," taken together, averaged 118. That, he thought, was very interesting as showing how small was the difference between the rolled irons and the steels. That was a point which was also referred to in the Paper.

TABLE XXXVII

AVERAGE GENERAL CORROSION FOR HALF-TIDE AND COMPLETE-IMMERSION CONDITIONS OVER 5 AND 10 YEARS. COMPILED FROM TABLE XXIV OF THE SEA ACTION COMMITTEE'S XVTH REPORT. WROUGHT IRON "N" TAKEN AS 100.

Material.	Mark.		Relative corrosion.	Remarks.
36-per-cent. nickel steel	L		36	
13½-per-cent. chromium steel	J		51	Severe pitting.
3½-per-cent. nickel steel	K		72	
Mild steel with ½ per cent. copper	G	90	95	
Mild steel with 2 per cent. copper	H	99		
0·4-per-cent. carbon steel	D		108	
Medium carbon steel, with low S & P (scale on)	E	114	112	Considerable pitting.
Mild steel with 0·7 per cent. manganese (scale on)	F	111		
Ingot iron (scale removed)	M	121	112	
Wrought iron (scale removed)	N	100		
Swedish charcoal iron (scale removed)	P	115		
Medium carbon steel with low S & P (scale removed)	A	135	129	Moderate pitting.
Mild steel with 0·7 per cent. manganese (scale removed)	C	123		

In that connection he might say that, when he first came to London, about 30 years ago, engineers used to fight very shy of steel for jetties, piers, or similar structures in sea-water. Later on, when the use of steel greatly increased and wrought iron was more difficult to obtain, sleeves of wrought iron were placed over the piles where the corrosion would be worst, and the space between filled up with concrete. That was an expensive arrangement, was troublesome to carry out and interfered somewhat with the fixing of the bracing. Now, however, that precaution might seem unnecessary, because the steel was almost as good as the wrought iron in resisting corrosion.

There was a difference between the steels "E" and "F," from which the rolling-mill scale had not been removed, and the steels

"A" and "C," which had been scaled. The steels which were not cleaned had a figure of 112, whilst those cleaned had a figure of 129. As against that, although the general corrosion was less with the unscaled steel, the cleaned steel was much less liable to pitting and was therefore a more reliable metal to use. He did not know what it cost to clean steel properly and to remove the scale, but when that was done it was clearly of great benefit. Mr. Wilson.

He might refer to one point not mentioned in the Paper, but which was referred to in Dr. Newton Friend's report on which the Paper was based, and which affected the value of the cement-coating to the steel where it was buried in the concrete; the point might be of interest to those who used reinforced concrete. When the Committee sent out the bars originally, a difficulty was how to distinguish them after the experiments were finished. It was clearly of little use merely to stamp the bars, and so a series of holes was drilled in the top and bottom to indicate the number of the bar and the type of metal. The holes were drilled completely through the bars, and when the latter were to be exposed, they were fixed in concrete frames, the holes being buried in the concrete. When the bars were cleaned for examination the holes were found just as clean as though they had been recently drilled, showing that the steel had been completely protected by the concrete.

Mr. GEORGE ELLSON observed that reference had been made on p. 10 to the effect of corrosive action on the strength of metals. He could say from personal observation that those members of structures subject to the highest stress corroded the most readily. He agreed with, and would emphasize, the necessity in the design of metallic structures of allowing some margin above bare requirements, and also of avoiding members of a slender and a flimsy nature, using preferably members with as little exposed superficial area as possible. In other words, it was desirable to employ sectional material which was stout and rigid, and this was a point which was often overlooked. Mr. Ellson.

Another point to which he desired to make special allusion concerned the removal of mill-scale. On p. 21 it was stated that steel with its scale left on had the advantage in fourteen cases out of twenty-four under corrosion tests. Later in the Paper, however, allusion was made to the question of the pitting of iron, it being stated that the specimens had had the advantage of having had their scale removed, while a remark was also made as to the advantage of cleaned steels over the steels which had their mill-scale untouched. The Authors had also commented on the beneficial effect on the pitting of ordinary iron and steel produced by previously removing the scale. Those remarks appeared somewhat contradictory, and he was unable to follow them in view of the first state-

Mr. Ellson. ment which he had quoted, namely that in fourteen cases out of twenty-four the advantage was with steels in which the mill-scale had not been removed.

His own experience was that it was entirely injudicious and harmful to remove the original scale of the metal. The fallacy of the benefits of removing the scale had persisted for a number of years. Some 30 years ago, when the renovation of the roof of Cannon Street station came under his personal supervision, a certain amount of rust had taken place on some of the members, but he had noticed that in every case where identification marks had been painted on to the skin of the metal at the works they stood up in relief, and in no case had corroded. To stop the corrosion, the surface of the metal had to be almost polished bright, and this was done at a very heavy cost. If corrosion were once allowed to commence, either by the pickling process to remove the mill-scale, or from any other cause, it was difficult and costly to remove the rust in order to get the protective coat of paint on to the metal itself, and, if the paint was put over any rust, its protective qualities would be heavily reduced, as the corrosion would still go on underneath the paint. His own practice was to specify a coat of red-lead paint to be put on at the maker's works, thus protecting the skin of the metal. After erection it was wire-brushed and given the final coats of paint, generally three in number. In that process a small amount of mill-scale was removed, but very little, and a clean surface was obtained on which to put the final coat, with a very satisfactory general result.

In the case of Charing Cross bridge, which was completed in 1863, the original paint was put on what was known as the mill-scale, or, as he preferred to call it, the original skin of the metal. The paint-work had been removed a number of times during renovation-work, but the skin of the metal was still as good on the great majority of the area as it was on the day when the bridge was completed. Similar remarks applied to Cannon Street bridge and to the other river-bridges of the Southern Railway.

It was stated in the Paper that, of all the ordinary steels and wrought irons, steel "D," with its rather high sulphur- and phosphorus-contents, was the only one to contend effectively with a specially energetic pitting action such as occurred under half-tide conditions at Halifax. He hoped that no one would deduce from that that a high sulphur-content was in any way helpful in reducing corrosion. On the Southern Railway it had already been necessary to renew conductor-rails which were laid down in the years 1915 and 1916 and which should have had a life of upwards of 30 years; that was entirely due to rapid corrosion arising from a high sulphur-

content. A typical analysis showed that their composition was approximately as follows :— Mr. Ellson.

Carbon, 0·07 per cent.; manganese, 0·28 per cent.; silicon, trace; sulphur, 0·14 per cent.; phosphorus, 0·05 per cent.

At the present time, the conductor-rails, of which the Southern Railway employed many thousands of tons, were made of steel of which the sulphur-content was not allowed to exceed 0·04 per cent., and the corrosion of that steel was very small indeed.

A corrosion-test had been made recently in Sevenoaks tunnel of copper-bearing steel rails of the following analysis :—

Carbon, 0·61 per cent.; silicon, 0·16 per cent.; phosphorus, 0·0029 per cent.; sulphur, 0·038 per cent.; manganese, 0·10 per cent.; copper, 0·34 per cent.

At the end of nearly 4 years, consecutive lengths of that copper-bearing steel rail and of ordinary steel rails were taken up and weighed. The average loss of weight was found to be as follows: the copper-bearing steel had lost 7·5 lb. per yard, and the ordinary steel had lost 9·45 lb. per yard. That test, however, was carried out on only about 50 tons of rail, and, therefore, too much importance should not be attached to the result; but it seemed to confirm to some extent the results found in the various experiments which had been carried out by the Sea-Action Committee.

Under conditions where sulphurous fumes existed in the atmosphere, as, for example, in enclosed stations under steam-traffic conditions, he had found that the best protection was given by graphite coatings rather than by lead paints, and that was particularly noticeable in Cannon Street, Charing Cross, Victoria and Dover Marine stations. Coatings of coal-tar had also been found to be very satisfactory. He had referred to that question in a Paper[1] which he read in 1921, and in that Paper he gave particulars of the mixture used, which was still standard on the Southern Railway for coal-tar coatings, and gave excellent results.

He had read with considerable interest each of the Committee's reports as they had appeared, and had distributed them to all his divisional officers; he attached great value to them. At the same time, he wished to say, with all deference, that there was still, he thought, a great field for research in the matter of the protection of metallic structures, and in his view the value of the work would have been considerably enhanced if the co-operation of engineers who were concerned with structures had been obtained, so that data could be accumulated as to the best methods to be adopted under actual

[1] "Cannon Street Bridge Strengthening," Minutes of Proceedings Inst. C.E., vol. ccxi (1920–21), p. 305.

Mr. Ellson. working conditions. Such co-operation would, he was sure, have been willingly given, but he was afraid that it would be necessary to wait at least 20 years before anything of value was obtained. A record of the value of different protective coatings applied under suitable conditions at different places in varying circumstances would be very useful, bearing in mind the great amount of ordinary commercial steel which existed in metallic structures. That was where the investigations would be of value. It would appear that perhaps the most hopeful results, from an economic and practical point of view, for the reduction of corrosion had been those obtained from the copper-bearing steel. As a result of those tests, he had used about 1,000 tons, and was hoping for good results. He had not used any "Chromador" steel, but, according to the results given, it appeared to have very good non-corrosion properties. An investigation had been carried out under the auspices of the Iron and Steel Institute by Dr. W. H. Hatfield,[1] and Mr. Ellson thought that that pamphlet should be studied.

The general practice on the Southern Railway for the maintenance of metal structures consisted of giving two coats of coal-tar, wherever it was practicable and æsthetically permissible. For the bulk of the remainder, a reliable lead paint was employed. The Southern Railway manufactured its own lead paints, but he did not mean to imply by that statement that it was not possible to buy reliable lead paints, because, in fact, very excellent lead paints could be bought. In other cases, zinc-oxide base-paints were used, and a certain amount of experimental work with lithopone had been done, but nothing had been found which was an improvement on the standard practices which he had previously mentioned. In terminal stations, subject to sulphurous atmosphere, graphite paints were used. For new works, wherever it was possible to do so the steelwork was embedded in concrete, and in other cases the exterior of the steelwork was as far as possible also encased in concrete.

Dr. Hudson. Dr. J. C. Hudson congratulated the Authors on their Paper, and particularly on their analysis of the corrosion-tests conducted by The Institution; those tests served a very useful purpose. The particular point to which he wished to refer concerned the relative corrosion-resistance of wrought iron and steel. He thought it possible that the recent results obtained by the Corrosion Committee of the Iron and Steel Institute and the British Iron and Steel Federation, to which Mr. Ellson had very kindly referred,[1] might throw a little additional light on the points raised by the Authors.

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[1] "The Work of the Corrosion Committee," Special Report No. 11, The Iron and Steel Institute, London, 1936.

Dr. Hudson.

He would like to make it clear that when, in the course of his remarks, he referred to "the Corrosion Committee" he would be referring to the Corrosion Committee of the Iron and Steel Institute, and not to the Sea-Action Committee.

There could be no doubt that the Authors' conclusion that, of the ordinary rolled materials, the Low Moor iron was distinctly the best as regards general wastage was perfectly justified by the experimental data, more particularly perhaps in the intermittent and the total-immersion tests. So far as the relative behaviour of wrought iron and mild steel was concerned, the researches of the Corrosion Committee were tending to show that any difference between those materials was due to one or both of two factors: firstly a difference in the character of the rolling-mill scale, and secondly a difference in macrostructure.

With regard to the first point, it had been found in the research-work of the Corrosion Committee that when wrought iron and steel were exposed side by side to the atmosphere, the rolling-mill scale adhered much more tenaciously to the wrought iron. A specimen of Scottish wrought iron had been exposed to the atmosphere of Llanwrtyd Wells for 1 year, and at the end of that period only 21 per cent. of the scale on the front had been removed by weathering. Even after the specimen had been vigorously scratch-brushed, all the loose scale being removed, 55 per cent. of the mill-scale was still adhering to the front of the specimen, and had afforded complete protection over that area for 1 year. That result had been confirmed by observations made at other stations selected by the Corrosion Committee; in the tests on as-rolled specimens at Sheffield, the loss in weight on exposure for Staffordshire wrought iron was 58 grams,[1] whilst the Scottish wrought iron lost 15 grams, and ordinary commercial mild steel 148 grams. After the scratch-brushing (that was, the removal of loose scale and rust), the figures were 76 grams for the Scottish wrought iron, 91 grams for the Staffordshire wrought iron, and 159 grams for the mild steel. It was the practice of the Committee to remove all rust from its specimens by pickling them in the presence of an inhibitor in an acid-bath. That placed the wrought irons at a disadvantage, because the loss observed naturally included the mill-scale which was still adhering to the specimens; but, in spite of that, the figure for the Staffordshire wrought iron was 179 grams, for the Scottish wrought iron 201 grams, and for the ordinary mild steel 214 grams. It was interesting to note that a much purer material, Swedish wrought iron made in Lancashire, lost 331 grams, and was, in fact, the most corroded material of those

[1] The losses in weight referred to specimens measuring 15 inches by 10 inches by $\frac{3}{8}$ inch, freely exposed to the atmosphere for 1 year. A loss of 100 grams was equivalent to 0·0024 inch of metal.—J. C. H.

Dr. Hudson. tested. The beneficial effect of copper, which had already been referred to by Sir Robert Hadfield, was shown; it reduced the loss of Swedish Lancashire iron from 331 to 176 grams. Those two irons were identical; he had seen them made on behalf of the Committee, and the only difference was the copper-content. The corrosion of mild steel, ordinarily 214 grams, was reduced to 167 by the presence of 0·2 per cent. of copper and to 165 by 0·5 per cent. of copper. Those three steels were absolutely identical apart from the copper contents, and were made from the same cast. There did not appear to be any marked difference in the effect of 0·2 and 0·5 per cent. of copper, which was an interesting and practical observation.

Those results showed that the rolling-mill scale on British wrought irons (but not on the Swedish), certainly afforded the metal some protection, at any rate during the early stages of exposure. Wrought iron was rolled at what was relatively a very high temperature, starting at about 1,500° C., and at that temperature the slag in the metal was liquid. It seemed probable that that slag was smeared over the surface during rolling, and constituted a very crude form of enamel. In confirmation of that view, the fact that the Swedish Lancashire iron, which contained very little slag, had behaved rather worse than the mild steel would tend to show that it was the scale, or the slag and scale, which was primarily responsible for that difference in the corrosion of the wrought iron and the mild steel over the short period of 1 year.

The Authors had stated on p. 35 that the superiority of the Low Moor wrought iron was not due to the fact that the rolling-mill scale was removed from it before test. He thought it might be said that the superiority was manifested in spite of the fact that the scale had been removed. That was, perhaps, rather an important point, because if that protective scale were removed from the wrought iron a factor was introduced which would not obtain under normal commercial conditions; such structural materials were used as they came from the mill, and therefore the wrought iron had the rolling-mill scale on it.

The second possible difference between the two materials lay in their macrostructure. Wrought iron, at any rate in the case of British iron, contained a large number of slag particles oriented in the direction of rolling. Those originated from the piling process, and also from the puddling process itself. According to a conception developed by Dr. Ulick R. Evans, it was possible that those slag particles acted as a mechanical barrier, and tended to obstruct the penetration of corrosion into the material. That was an interesting point, and, although he believed that Dr. Evans had recently modified his views to a slight extent, the Corrosion Committee had that theory

under test in various ways. In particular, they had prepared a wrought-iron wire from a pile made by building together wrought-iron sheets; the pile was then rolled down to a wire rod and drawn into wire. The size of the pile was about 4½ inches square. The theory was that all the slag particles resulting from the piling and from the manufacturing process would be arranged tangentially, so that the final wire would have the structure of an onion and would produce maximum obstruction to corrosion. If the theory was correct, that wire would give good results; if it did not, it would be necessary to change the theory. In a year or two results would be obtained from tests on that type of wire. Dr. Hudson.

There was one other matter which might be of general interest in illustrating the effect of rolling-mill scale. Ingot iron was a peculiar material, inasmuch as there was a brittle range between approximately 900 and 800° C., where it developed a tendency to brittleness, and it was not practicable to roll it. It was, therefore, necessary to complete the rolling of ingot iron above a temperature of 900° C., or alternatively to roll it partly above that temperature and then to allow it to cool through the brittle range before finishing the rolling. It was interesting to find that the normal scale produced at the high temperature would adhere to the specimen for from 3 to 6 months, whereas the scale produced at the reduced rolling temperature would be entirely removed in the space of from 2 to 3 weeks. The normal rolling-mill scale with the process finished at 1,060° C., was about ten times as thick as the low-temperature scale, when rolling was finished at 780° C.

Dr. Frank Wormwell said that Dr. G. D. Bengough regretted his inability to be present that evening, and had asked Dr. Wormwell to read a joint statement which they had prepared. Dr. Wormwell.

The question of the estimation of the amount of corrosion was discussed on p. 19. The results were originally stated by Dr. Friend as grams of metal lost per 1,000 square centimetres of surface. Dr. Wormwell considered that that did not give a true idea of the penetration of corrosion, even when converted by the means proposed in the Paper. To obtain a fair idea of the average depth of penetration the area of the metal actually corroded should have been measured, and the loss of weight divided by that area; such a measurement was often possible, especially in sea-water, and with carbon steels and wrought irons. When corrosion was mainly in the form of severely localized pitting, as, for example, with some alloy steels, the area-measurement would be difficult, and the behaviour of the material would be best expressed, as the Authors had said, by measurement of the number and depth of the pits.

The next point to which he desired to refer was the corrosion of

Dr. Wormwell. composite specimens; that was, the coupling of chromium steel with an ordinary mild steel or with cast iron. It was really necessary to consider one or two general principles which applied in those cases. He was considering metals completely immersed under sea-water at appreciable depths. A mild-steel test-piece of given area corroded at a definite rate, chiefly determined by the rate of oxygen-supply to the whole specimen. If, however, another specimen were taken of the same total size, but copper-plated over half its area, the total corrosion in a given time would be the same as if the whole specimen were bare, because the catchment-area for oxygen was unaltered; the amount of corrosion per square centimetre of the bare steel would, therefore, be approximately doubled. Thus, the increased initial electromotive force due to the presence of copper would have no effect on the total amount of corrosion, which would be determined by the catchment-area for oxygen. The copper might be regarded either as a hydrogen or as an oxygen electrode, its potential depending on the rate of supply of oxygen. If a specimen, half of which was covered with scale instead of copper, were considered, the result would be similar if the scale were a good electrical conductor; if, however, it were a bad conductor, the rate of corrosion per unit area of the base metal would be less than double that from the scale-free specimen. Dr. A. McCance, in a recent paper,[1] had expressed the view that scale on steel could be regarded as a non-conductor of electricity, and there seemed some support for that view. For example, in the present Paper it was shown (p. 21, and Appendix III, Table IX) that steels exposed with mill-scale suffered less total corrosion or wastage than steels exposed in a scale-free condition, whereas if the scale were a very good electrical conductor the total corrosion should have been the same both for the scale-covered and scale-free specimens. The total amount of corrosion on a test-piece was not generally directly proportional to its area, owing to the diversion of the oxygen-conveying convection-currents into different paths when the size of the specimen was changed. In order, therefore, to show correctly the effect of contact between dissimilar metals, the size of the composite specimen had to be identical with the size of the single specimen of each metal, a condition frequently overlooked in practice.

Those principles led to the conclusion that the contact of dissimilar metals was serious only when one of them was relatively resistant to corrosion, and was of large area relative to that more rapidly attacked. Accordingly, Dr. Wormwell had re-arranged

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[1] Dr. A. McCance, "The Corrosion Problem in Steel," Trans. Inst. Engineers and Shipbuilders in Scotland, vol. 79 (1935–36), p. 329.

Table XXVI (Appendix V) of the Paper, as shown in Table XXXVIII, Dr. Wormwell.
on the assumption that the "contact specimens" were made up of two single specimens of similar size.

TABLE XXXVIII.

CORROSION OF CHROMIUM STEEL (SOFT GRADE) BY RIVER-WATER, WHEN IN CONTACT WITH MILD STEEL OR CAST IRON. (DATA FROM APPENDIX V, TABLE XXVI, OF THE PAPER.)

Col.	Loss in weight: milligrams. (1) Material exposed alone.	(2) Calculated for double size.	(3) Actual total loss of dual specimen.	(4) Values for separate metals added together.	(5) Ratio (3):(4).	(6) Ratio (3):(1).
	Chromium steel 181	362	—	—	—	—
	Mild steel 853	1,706	1,363 + 35 = 1,398	1,034	1·35	1·64
	Cast iron 961	1,922	1,456 + 10 = 1,466	1,142	1·28	1·53
	Mild steel and cast iron in contact.					
	Mild steel 853	1,706	782 + 829 = 1,611	853+961 =1,814	0·89	1·89
	Cast iron 961	1,922				1·68

Considering first the upper part of the Table, which referred to chromium steel in contact with either mild steel or cast iron, it would be seen that the actual experimental results of Col. (3) were less than the calculated values of Col. (2), no doubt because doubling the size of test-piece did not necessarily double the amount of corrosion, as Dr. Bengough and himself had shown in 1933. Col. (6) showed that the corrosion of the composite specimen was either 1·64 or 1·53 times that of the single specimen. The reason why the values of Col. (4) were lower than those of Col. (3) was merely that the chomium steel separately did not use as much of the available oxygen as did the mild steel. When the metals were placed in contact the oxygen reaching the chromium steel became available for assisting the corrosion of the mild steel or cast iron, the latter being the anode and the chromium steel the cathode of an electrolytic cell. The total corrosion in the system was therefore increased either 1·35 or 1·28 times by placing the metals in contact, as was shown in Col. (5).

In the tests with mild steel in contact with cast iron, the lower value of Col. (3) as compared with Col. (4) was due to the fact that doubling the size of the specimen did not double the total oxygen-supply. That was shown by the ratio in Col. (5) being less than unity. Col. (6) showed again that placing two specimens in contact gave less than double the corrosion of a single specimen.

Dr. Wormwell. The three-dimensional diagrams of *Figs. 4*, *5*, and *6* of the Paper showed that the effect of a particular location was much less marked for metals completely immersed in the sea than for metals exposed to aerial attack. It was very satisfactory to read the statement on p. 47 that "Apparently the corrosive action of salt water is such as to render the constituents, structure and physical conditions of ferrous materials much less important than they are under atmospheric conditions," as that was precisely the conclusion that Dr. Bengough and himself had reached in their laboratory tests at the Chemical Research Laboratory, Teddington, on different ferrous materials, such as a highly-purified iron, acid and basic mild steels, 0·5-per-cent. copper steels, wrought iron, 0·9-per-cent. carbon steel, badly segregated steels containing 0·95 and 0·235 per cent. sulphur respectively, and a 5-per-cent. nickel steel. Their tests were made in all cases on specimens which were free from scale and which had been given a finely-turned surface, and were conducted in solutions of N/2 sodium chloride, N/10 potassium chloride, Teddington tap-water, and natural sea-water, and under atmospheres of oxygen or ordinary air. The results were set out in detail in Section D, Part 3 of the Third Report of the Corrosion Committee of the Iron and Steel Institute.[1]

The Authors' conclusions on the behaviour of copper steels appeared rather surprising in one respect. On pp. 37 and 38 it was stated that 0·5 per cent. of copper was beneficial both as regards general wastage and pitting, even in conditions of complete immersion. At first sight that seemed to differ both from the results of American tests[2] and from the laboratory tests in N/2 sodium chloride conducted by Dr. Bengough and himself. An examination of the Authors' comparison between copper steels and similar carbon steels (p. 37 *et seq.*) showed that they used steel "D," with 0·40 per cent. carbon, as a comparison-material. That was done because steel "D," according to the Authors, "is, on the average, the best of the carbon steels as regards wastage, and better than "F" as regards pitting. . . ." That statement appeared to be based on the average figures for pitting for all four ports and all three conditions of exposure, given in Appendix III, Table XVI. The figures given there were 2·16 for "D" as against 2·19 for "F," which was probably not a significant difference. Dr. Wormwell would suggest, however, that it was best to consider the relative merits of different materials for each exposure-condition separately. He only wished to discuss

[1] Published in 1935.

[2] V. V. Kendall and E. S. Taylerson, "A Critical Study of the A. S. T. M. Corrosion Data on Uncoated Commercial Iron and Steel Sheets," Proc. Am. Soc. Test. Mat., vol. 29 (Part II, 1929), p. 204.

Dr. Wormwell.

the results for complete immersion, which were comparable with the laboratory tests carried out by Dr. Bengough and himself; referring to Appendix III, Table XVI, for total-immersion conditions the average pitting for all four ports for steel "D" was 3·19 as against 2·58 for steel "F," so that it would appear better to take steel "F" as a basis of comparison for the effect of copper in conditions of total immersion. Again, from Appendix IV, Table XX, the corresponding figure for 0·635-per-cent. copper steel "G" was 2·64, practically identical with that for the ordinary steel "F." As regards general wastage, however, the copper steel "G" was superior (0·420 millimetres loss) both to steels "F" (0·487) and "D" (0·500) in totally immersed conditions, as well as on the average for all conditions, as had been stated by the Authors on p. 38. The Authors stated on p. 65 that in their tests in the Gulf of Paria on steels which were exposed, like the mild steel, with their scale untouched, the 0·29-per-cent. copper steel No. 3714 "again shows rather less wastage than the mild steel. In the present instance the copper steel has also shown a propensity to localized attack." Those results might be due to the better adherence of scale to the particular copper steels tested than to the ordinary steels; whether, however, that was a general property of copper steels had yet to be shown. Laboratory measurements made by Dr. Bengough and himself threw no light on the matter, since scale-free materials were used. In the tests of the American Society for Testing Materials[1] the specimens were exposed with mill-scale in situ, and for those total-immersion tests in Severn river-water (brackish), Washington city-water and Calumet mine-water, Kendall and Taylerson deduced by statistical analysis that the influence of small additions of copper, up to 0·7 per cent., had a negligible effect on the resistance to corrosion. The latter tests also agreed with the tests performed by Dr. Bengough and himself and with those of the Sea-Action Committee in showing that the effect of up to 0·08 per cent. of sulphur has no appreciable effect on the corrosion of steel under conditions of total immersion. It could not, therefore, be too strongly emphasized that, for steels exposed to aerial attack, additions of copper up to 0·5 per cent. had been conclusively shown to be beneficial. That fact emerged clearly from the work of the Sea-Action Committee and was supported by other exposure tests in Great Britain, the United States, and Germany. The difference between the response of materials to aerial exposure and immersion under water was very important, since it meant that conclusions from one set of data could not be applied to the other. For that reason it was often dangerous to

[1] Footnote 2, p. 116.

Dr. Wormwell. compare different materials from the average results for all three conditions of exposure. The results for each condition should be analysed separately.

Dr. Vernon. Dr. W. H. J. VERNON said that the large amount of valuable information which the Authors had set out in their Paper was clearly a tribute both to the planning of the main investigation under the Corrosion Committe of The Institution and also to the thoroughness with which the experimental part of the work had been carried out by Dr. Newton Friend. The wideness of the investigation had inevitably led to a vast amount of data, and in making an independent survey, the Authors had undoubtedly performed a valuable service. The use of three-dimensional diagrams, which brought out very clearly the relationships between corrosion, climatic conditions and the materials under examination, was particularly helpful.

The Authors' use throughout the Paper of the term "aerial conditions" was likely to give rise to some confusion, even although it was difficult to see what other term they could have used to distinguish such conditions from those of partial or complete immersion. In all cases in the main investigation the specimens had been exposed in very close proximity to the sea. Inasmuch as the work was being carried out for the Sea-Action Committee clearly no exception could be taken to that procedure. It should be emphasized, however, that such conditions were not typically aerial, and very different results might have been obtained had the specimens been exposed inland, even at only a comparatively short distance from the sea.

That point was of some importance, first because the corrosion values of a series of comparable specimens served to compare the corrosivity of the several atmospheres concerned, and secondly because differences in behaviour of various materials were brought out far more markedly, as the Authors had very rightly emphasized, under atmospheric conditions than under conditions of complete immersion. The second aspect was of much significance, but he would confine himself to remarks on the first.

The importance of defining clearly the conditions of exposure when assessing the climatic factor was clearly seen by comparing the present results with those which had been recently published by the Corrosion Committee of the Iron and Steel Institute.[1] In both cases the available data had permitted the different corrosion-stations to be placed in a fairly definite order of merit, but in so far as the stations were representative of comparable climates that order differed very materially in the two investigations. In the Iron and Steel Com-

[1] Dr. J. C. Hudson, "The Committee's Field Tests on Atmospheric Corrosion." Third Report of the Corrosion Committee (1935), Section B, p. 49.

mittee's research the stations fell, with only one exception, into two groups; those comprised six overseas stations, of relatively low corrosivity, and five stations in Great Britain, of relatively high corrosivity. The first group included such widely differing climatic conditions (particularly as regards temperature) as Abisko (within the Arctic circle), Khartoum, and Singapore. The inclusion of Singapore in the group of relatively low corrosivity (Llanwrtyd Wells and Calshot gave corrosion values approximately four times greater, and Sheffield nearly eight times greater) was rather remarkable, in view of the fact that Colombo, which was on very nearly the same latitude and presented a very similar type of atmosphere as Singapore, was by far the most destructive of the atmospheres included in the Sea-Action Committee's investigation. In fact, on the question of climatic influences there was no correlation whatever between the two investigations. Dr. Vernon.

What then was the explanation? Some light appeared to be thrown on the matter by the examination of results recently obtained at Teddington.[1] When a specimen of iron was exposed to purified air, the relative humidity of which was increased gradually with time from zero to very nearly saturation-point, the rusting that occurred at the higher relative humidities was very small in amount and not at all serious in character. If, however, traces of impurities were admitted into the atmosphere, then very remarkable changes occurred. If, for example, the air contained traces of sulphur dioxide, primary and secondary critical humidities might be recognized. When the relative humidity reached the primary value (about 50 per cent. relative humidity), rusting commenced; when, however, the secondary critical humidity was reached, at about 80 per cent. relative humidity, the rate of attack suddenly increased in an extraordinary manner, and rusting proceeded rapidly.

It was important to observe that those effects might be reproduced in just the same manner if, instead of admitting gaseous impurities into the otherwise pure atmosphere, the specimen itself were "inoculated" at the outset with particles of solid impurity in very small amounts; particles of hygroscopic salts were very efficacious in that respect. Again, very great increase in the rate of attack was produced at the secondary critical humidity, an increase which appeared to be associated with the physical properties of the first-formed rust.

The bearing of those experimental results on the results of field tests was now apparent. So long as the climatic influences were confined either to differences of temperature or to differences of relative

[1] Dr. W. H. J. Vernon, "A Laboratory Study of the Atmospheric Corrosion of Metals," Part III, Trans. Faraday Soc., vol. 31 (1935), p. 1678.

Dr. Vernon. humidity (the atmosphere being otherwise pure) comparatively small differences in rates of rusting were to be expected. If, however, the specimens were situated in such a position that they could become "inoculated" with salt particles, then very great differences in the rates of rusting might be expected at the higher relative humidities. The rate of attack in a relatively pure atmosphere might then approximate to that produced in a polluted atmosphere in the absence of salt particles. The greatest rates of attack were to be expected in industrial districts, such as those in Great Britain, where the presence of both gaseous and solid pollution coincided with conditions of fluctuating and generally high relative humidity. Under tropical conditions, however, the influence of particles and humidity alone might outweigh the added effect of gaseous pollution under milder conditions of temperature. That appeared to be the case with the "aerial" tests at Colombo as compared with the one example of a British industrial atmosphere included in the present results (Birmingham, as shown in *Fig. 11*).

The results dealing with the influence of scale initially present on the specimen were particularly interesting. (Incidentally, the Authors' use of the term "scaled steels," to indicate steels from which the scale had been removed, was confusing, and Dr. Vernon would suggest that it should not be used. The word "descaling" had recently been criticized, but if the substitution of the word "scaling" were advocated, he would ask what term was to be used instead of "scaling" when that word was used, in its legitimate and more obvious sense, to imply the formation or acquisition of scale ?) The results showed, in most cases, the greatest general corrosion on those steels from which the scale was not originally removed. The maximum effect was observed at Plymouth; at Colombo there was practically no difference in the results. Presumably, the greater corrosion given by the steels with scale initially on was due to the de-scaling that usually occurred on exposure to the atmosphere, the loss of scale adding to the total loss of weight; there was a certain amount of undermining and the scale tended to peel off. No doubt the process was hastened by temperature changes, owing to the unequal coefficients of expansion of scale and metal. It was perhaps rather unfortunate that the copper-bearing steels were all exposed with scale on, because the beneficial effects of copper were probably directed, at least in part, toward modifying the properties of the first-formed rust on exposure to the air.

Finally, he would like to refer to Sir Robert Hadfield's remarks concerning the Delhi Iron Pillar, in respect to its notable freedom from corrosion. The results of the X-ray examination of the surface layer, to which Sir Robert had referred, would certainly be awaited with

Dr. Vernon.

great interest. Meanwhile, it might be pertinent to mention that, in the light of recent work, there was nothing inconsistent in that freedom from corrosion with the climatic conditions which the Delhi Pillar had enjoyed, more particularly in the earlier stages of its 1600 years' exposure. Those conditions involved a generally low humidity, a freedom from sulphur-dioxide pollution, and an absence of "inoculating" salt-particles. Under such conditions iron either did not rust at all (if the humidity were below the "critical humidity"), or, even if the critical humidity were exceeded, the rust which formed would be small in amount and actually protective in character. The presence of carbon dioxide, so far from increasing the rusting, actually depressed it.[1]

Earlier work [2] had shown that in the absence of particles (for example, when screened within a muslin "cage"), iron developed an invisible film of oxide which showed marked resistance to rusting on subsequent normal exposure. It had formerly been thought that that invisible oxide-film ceased to thicken soon after its formation, but recent work [3] had shown that such was not the case; on the contrary, a progressive increase in thickness occurred. The rate of attack certainly decreased with time (depending approximately on the relation $W^{2 \cdot 5} = kt$ in early stages where W denotes thickness of film, and t denotes time), but in the course of centuries the film would undoubtedly reach appreciable dimensions, and would almost certainly be sufficient to account for the resistance to mild conditions of atmospheric corrosion for which the Delhi Pillar was famous. Very probably a thin scale formed during manufacture would contribute its quota of protection, and the X-ray examination might decide whether that was the major factor; it might, however, be fairly safely surmised that even in its absence nature would still have produced, in course of time, an efficient substitute.

Mr. Manning.

Mr. H. D. Manning asked permission to refer to one point which, although not specifically mentioned in the Paper, might, he thought, be regarded as coming within the scope of the discussion. Engineers responsible for the design of sewerage and sewage purification-works would all be familiar with the problem of corrosion of such works. The subject was of increasing importance owing to the more widespread use of steel pipes for sewerage purposes and to the way in which modern purification-plants were becoming mechanized.

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[1] Footnote 1 on p. 119.

[2] Dr. W. H. J. Vernon, Second Report to the Atmospheric Corrosion Research Committee (British Non-ferrous Metals Research Association), Trans. Faraday Soc., vol. 23 (1927), p. 159.

[3] Dr. W. H. J. Vernon, "A Laboratory Study of the Atmospheric Corrosion of Metals," Part II, Trans. Faraday Soc., vol. 31 (1935), p. 1670.

Mr. Manning. The fact that the Ministry of Health would now sanction loans for 30 years on steel sewers had lessened, but had not removed, the doubts which were felt as to their ability to stand up to sewer-conditions. In that connection, it was necessary to distinguish between pumping-mains and gravity sewers. In the former the pipe was normally full of sewage under pressure, and it was to be assumed that the danger of internal corrosion was not great. In a gravity sewer, however, the water-level was constantly changing, and the ventilation was often poor. It was said that, in practice, a layer of grease formed over the sides of the pipe and protected the metal from attack. That might be the case, but it was curious that, in the case of concrete or brickwork sewers, where a similar grease-film might be expected, the cement was sometimes very seriously attacked on the top half of the arch and along the wind-and-water zone.

The principal causes of corrosion in sewers were presumably the damp conditions, coupled with the presence of such gases as sulphur dioxide, sulphuretted hydrogen, carbon dioxide and carbon monoxide. Possibly, also, the grease-film, instead of forming a protection, was actually a danger owing to the formation of fatty acids resulting from its decomposition. There was also the danger of external attack due to ground-waters containing sulphates in solution. Whilst it was common practice to protect the outside of steel pipes with bituminous wrappings, the danger of external attack could not be overlooked, especially if those wrappings were omitted. No doubt a great deal of independent investigation had been carried out on the subject, but, so far as he was aware, the published information was scanty.

With reference to the increasing use of steel in sewage-purification works, until recently it had been possible to make most of the fittings required on such works in cast iron, wrought iron, or gunmetal. To-day the average sewage-works included mechanical screens, cleaning mechanisms for sludging the tanks, and stirring mechanisms and gasholders for digestion-tanks. Such apparatus involved the use of large quantities of steel, and time had yet to show what the life of those structures would be.

The use of steel gasholders for sludge-gas was of particular interest. Gasholders containing ordinary town's gas were protected, he believed, by the formation of a tarry deposit on the internal surfaces, but that would not apply in the case of sludge-gas. He believed that that gas had been found to be most destructive of brass and copper fittings, such as meters and valves, more especially where water was present. In some large gasholders recently constructed for sludge-gas, use was made of copper-bearing steel

having a copper-content of about 0·25 per cent., in the hope of resisting corrosion. In the absence of practical results or comprehensive experiments, designers were greatly handicapped, and he would like to ask the Authors whether they could express any opinion as to the suitability of copper-bearing or other special steels for purposes such as he had mentioned. The importance of the subject might be realized when it was stated that, on a typical modern sewage-purification works, about 25 per cent. of the cost might be debited to machinery and steelwork as distinct from normal constructional work in concrete and brick. The Paper showed the complex nature of marine corrosion, and it might well be that the conditions arising in sewers and on sewage-disposal works were even more varied than marine conditions. He felt sure, however, that the results of a full investigation of the problem would be of very great value. Mr. Manning.

Mr. W. R. Barclay expressed his thanks, as President of the Institute of Metals, for having been invited to attend the meeting. Although the Institute of Metals was mainly concerned with the non-ferrous metals, corrosion was one of those general subjects with which both sections of metallurgical science were concerned. Mr. Barclay.

Sir Robert Hadfield thanked the speakers for their kind comments on the Paper. It might be of interest to mention that no one had yet succeeded in producing a ferrous alloy of the same high resistance to corrosion as Michael Faraday had done. Faraday had made an alloy of 50 per cent. rhodium and 50 per cent. iron which seemed practically incorrodible under almost any conditions; Sir Robert had described it fully elsewhere.[1] He had desired to recommend the inclusion of some rhodium-iron specimens in the experiments of the Sea-Action Committee of the Institution, but had found that each bar would cost something like £1,600, so that it was not possible to pursue the matter further in that direction. Sir Robert Hadfield.

The Authors, in reply, thanked Mr. M. F-G. Wilson, the present Chairman of the Sea-Action Committee, for his very kind remarks regarding the help which one of them (Sir Robert Hadfield) had tried to give to the investigations of the Committee since its commencement 20 years ago. Sir Robert had always found it a great pleasure to do anything he could to assist the work of the Committee. The Authors.

They agreed with Mr. Wilson as to the usefulness of a percentage basis in comparisons of the various materials, and had themselves found useful the comparisons made in that way in the Committee's Reports. The method adopted in the Paper had been used rather as a means for conveniently and systematically carrying

[1] "Faraday and his Metallurgical Researches," pp. 214 and 216. London, 1931.

The Authors. out the examination made of the influences on corrosion of the different climates concerned, and of the different conditions of exposure.

With regard to steels containing copper, the results of the Committee were, as had been stated by some speakers, in agreement with those of other past and current researches, and with the practical experience so far obtained with those steels. The present research had, it was believed, gone further, in showing that for marine conditions the beneficial addition of copper might extend beyond the small amount, namely about 0·3 per cent., which appeared to be the limit of useful addition for steels exposed to ordinary atmospheric conditions. The Committee had thus been justified in its decision to include steels containing larger percentages of copper, namely 0·6 and 2·2 per cent.

Copper steel was undoubtedly of value in contending with corrosion, but its choice for any particular purpose should be the result of careful consideration of its capabilities; while no dangers seemed in any case to be attached to its use, it was only in particular circumstances that copper steel offered advantages. In sea-water 0·50-per-cent. copper steel was worthy of consideration for practical trial, as had been suggested by Mr. Wilson. Such practical experience was the only way to determine the merits of copper steel, and the same was true of the steel containing 36 per cent. nickel, referred to by Mr. Wilson.

A subject which had been referred to by several speakers was the influence of the previous removal of scale. The direction, favourable or otherwise, of that influence depended on circumstances. There was not necessarily any conflict between the experience in that respect of Mr. Ellson and the Committee's results referred to in the Paper; an important point of difference was that Mr. Ellson was concerned with painted structures. Dr. Hudson's results referred, as did those in the Paper, to unpainted steel, and the two series were in marked agreement. The most marked influence of scaling was, however, shown, not under atmospheric conditions, but in its favourable effects on the pitting which occurred in sea-water. There was undoubtedly room for a closer study of the bearing of the actual character of rolling-mill scale on its influence as affecting corrosion, and the results in that direction provided by Dr. Hudson were of decided interest.

Mr. Ellson's remarks were of much practical value; the Authors agreed with his suggestion that there was still a great field for research in the matter of the protection of metallic structures. Naturally, the subject was complex, dividing itself into (*a*) the type of steel employed, (*b*) its heat-treatment, as the same steel under

different conditions might give quite different results as regards corrosion, (c) the method of manufacture and the care taken in its production into finished forms. Mr. Ellson had pointed out that to determine corrosion-qualities thoroughly required a long time; the researches of the Sea-Action Committee were coming to a close, but those of the Iron and Steel Institute and other bodies would be continued for many years, and would probably help to provide an answer to Mr. Ellson's queries. The Authors.

The Authors were glad to have the confirmation provided by the work of Drs. Bengough and Wormwell as to the levelling influence of actual exposure to sea-water on the relative merits of different ferrous materials; that was one of the most noticeable features both of the Sea-Action Committee's and of the Authors' own results. In particular, the behaviour of Steel "B," with a high sulphur and phosphorus content, was especially striking, and left no doubt that the effects of those impurities in ordinary steel were much less harmful in sea-water than in air. The point, however, was for the present one rather of scientific interest, and the Authors would be the last to advocate the use of impure steel. Further, under marine aerial conditions steel "B" had been shown to be definitely inferior to purer steels, thus agreeing with Mr. Ellson's experience with conductor-rails; presumably, however, the latter were mostly exposed in a rather different kind of atmosphere.

In the large majority of cases the Sea-Action Committee's specimens were corroded over their whole area between the concrete supports, and the method adopted of assessing penetration would thus for the most part appear to be identical with that suggested by Dr. Wormwell. The exceptions were mainly provided by the chromium steels, which were subject to severe localized pitting and for which, therefore, the method proposed would be unsuitable.

The description by Dr. Wormwell of the principles underlying the corrosion of dissimilar metals in contact was of great interest. Contact-effects could not altogether be avoided even with the use of ordinary materials, and they were responsible for many cases of excessive corrosion. By the intelligent application in practice of the knowledge afforded by fundamental research, it should be possible to minimize such effects with advantage.

The adoption of different carbon steels in each separate type of exposure as a basis of comparison for the performance of the alloy steels would have introduced an undesirable complication into the Tables of results. It was doubtful, too, whether such a course would be justified, since true comparison could only be on the basis of composition, in which respect steel "D" was chosen by the Authors as being the best. Desired comparisons in other ways

The Authors. could, in any case, readily be made, as had been shown by Dr. Wormwell. The Authors' conclusions that the copper steels were subject to pitting to about the same extent as carbon steels, while their total wastage was less, appeared, however, to be correct even for under-water conditions if comparison were made alternatively with steel "F."

They were especially interested in Dr. Vernon's remarks, which did much to justify their own prediction in the Paper as to the eventual importance in the solution of the corrosion-problem of the kind of fundamental research in which Dr. Bengough and his colleagues were engaged. It was apparent that connections were now steadily being established between the results of laboratory research and field tests systematically conducted, and through them with practical experience. One of the Authors (Sir Robert Hadfield), having visited Singapore, would not have been surprised if the climatic conditions there had been found to be particularly corrosive to iron; Dr. Vernon, however, threw considerable light upon why the corrosion there was actually relatively low in amount as compared with that at Colombo, which had a similarly hot climate. Dr. Vernon's observations regarding the Delhi Iron Pillar, especially as to the possible part being played by a protective film in its freedom from corrosion, were particularly interesting and were valued as showing that modern scientific knowledge at any rate did not find it necessary to suppose any special virtues in the material of which the pillar was made which were not shared by other high-quality wrought irons.

In conclusion, the Authors would like to say how much they appreciated the information provided by Messrs. Wilson, Ellson and Manning, and based on their practical experience of corrosion. In presenting their Paper the Authors felt that its chief purpose would best be served by the collaboration in discussion of practical engineers and those interested in the scientific study of corrosion; they trusted that the written contributions to follow would show the same gratifying results in that respect as the discussion had done.

A large selection of specimens, from both the Sea-Action Committee's and the Authors' researches, was shown at the Institution during the meeting.

*** The Correspondence on the foregoing Paper will be published later.—Sec. Inst. C.E.

ORDINARY MEETING.

21 April, 1936.

Mr. JOHN DUNCAN WATSON, President, in the Chair.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

"The Rational Design of Steel Building Frames."

By Professor JOHN FLEETWOOD BAKER, M.A., D.Sc., Assoc. M. Inst. C.E.

TABLE OF CONTENTS.

INTRODUCTION.

FOR more than 30 years the steel frame has occupied an increasingly important place in building-construction. As far back as 1909 regulations were drawn up governing its use in London. These regulations, contained in the London County Council (General Powers) Act, 1909, not only served as the model on which were based the Codes of Practice governing the use of steel in buildings of almost all other countries in the world, but remained in force for 23 years. Long before the expiration of that period, the development in the manufacture of steel and the advance in the technique of steel-work made engineers feel that the regulations were unduly restrictive and that they did not allow full advantage to be taken of the excellent qualities which steel possesses as a material for building-construction. It was in response to this feeling that the British Steelwork Association approached the Department of Scientific and Industrial Research with a request that investigations should be instigated; they also offered funds towards the cost of such investigations. The offer was accepted and the Steel Structures Research Committee was appointed in August, 1929, having as members representatives of The Institution and of the British Steelwork Association, together

with other practising engineers and nominees of the Department. The terms of reference were as follows :—

(*a*) To review present methods and regulations for the design of steel structures, including bridges.

(*b*) To investigate the application of modern theories to the design of steel structures, including bridges, and to make recommendations for the translation into practice of such of the results as would appear to lead to more efficient and economical design.

The Committee was in existence for more than 6 years, and in that time it considered more than one hundred and fifty reports from investigators. The more important of these, together with recommendations for the design of steel structures, are contained in the First, Second and Final Reports of the Committee published in 1931, 1934 and 1936.[1] These Reports, although they do not contain all the details of the investigations carried out for the Committee, form a considerable literature dealing with the behaviour of steel frames. The reports cannot be appreciated without considerable study, and it is thought, therefore, that the outline of the Committee's work, which will be given here, may be of some value to the engineer wishing to appreciate the complete investigation before studying the details contained in the published reports.

Review of Existing Methods of Design.

It was found impossible in the time available to consider steel bridges in detail, and the first task undertaken by the Committee was a review of the regulations governing the design of steel-framed buildings throughout the world. This showed that, although there were some striking discrepancies in detail, the method of design implied in every case was the same. The differences in detail are no longer of interest to the engineer practising in this country, since as an outcome of the Committee's First Report a uniform Code of Practice is now adopted by almost all authorities in Great Britain, and in considering the design-method used to-day this Code alone need be studied.

So unsatisfactory at the time of the Committee's inception were the rules governing the use of steel in buildings that it was felt desirable, in view of the considerable period which would elapse before the results of the full research-programme were available, to draw up recommendations for a Code of Practice, based on

[1] Published by His Majesty's Stationery Office.

knowledge existing then, which would remove many restrictions. It must be emphasized that these recommendations were not based on any investigations carried out for the Committee, but on the available knowledge and experience of practising engineers. These recommendations, made by the Committee in its First Report, have been accepted practically unchanged by the London County Council as the basis of consideration of applications for waiver under Section 58 of the London Building Act, 1930, for relief from the provisions of the Third Schedule to that Act, and have been embodied in the British Standard Specification No. 499 issued in April, 1932, by the British Standards Institution; they have also been recommended to local authorities by the Ministry of Health, and a study of them will show the method of design in general use to-day.

The relevant clauses are as follows :—

(12) The steel framework of a building, in combination with the floors, bearing walls and bearing structures (if any) and the foundations shall be capable of safely and independently sustaining the whole dead and superimposed load of the building together with all forces due to wind, earth or other pressures acting upon it.

(35) (*a*) The dead load of a building shall consist of the actual weight of walls, floors, roofs, partitions and all other permanent construction comprised in such building, etc.

(*b*) The superimposed load in respect of a building shall consist of all loads other than the dead load.

(*c*) For the purpose of calculating the loads on foundations, pillars, brick or stone piers, walls and beams in buildings, the minimum superimposed load on each floor and on the roof shall be estimated as equivalent to the following dead load :—

	lb. per sq. ft. of floor area.
Rooms used for domestic purposes, hotel bedrooms, hospital rooms and wards	40
Offices, floors above entrance floor	50
Offices, entrance floor and floors below entrance floor	80

etc., etc.

(*e*) For the purpose of calculating the total load to be carried on foundations, pillars, brick piers and walls in buildings of more than two storeys in height the superimposed loads for the roof and topmost storey shall be calculated

in full in accordance with the schedule of loading, but for the lower storeys a reduction of the superimposed loads may be allowed in accordance with the following table :—

Next storey below topmost storey . .	10 per cent. reduction of superimposed load.
Next storey below	20 per cent. reduction of superimposed load.
Next storey below	30 per cent. reduction of superimposed load.
Next storey below	40 per cent. reduction of superimposed load.
All succeeding storeys	50 per cent. reduction of superimposed load.

The above reduction may be made by estimating the proportion of floor area carried by each foundation, pillar, pier and wall. No such reductions shall be allowed on any floor scheduled for an applied loading of 100 lb. or more per sq. ft.

(*f*) All buildings, other than those indicated below, shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than 15 lb. per sq. ft. of the upper two-thirds of the vertical projection of the surface of such buildings with an additional pressure of 10 lb. per sq. ft. upon all projections above the general roof level. On the sea coast and in similarly very exposed situations a further provision not exceeding 10 lb. per sq. ft. shall be made.

If the height of a building is less than twice its average width, wind pressure in general may be neglected, provided that the building is adequately stiffened by floors and walls.

(36) (This clause contains particulars of the permissible working stresses for steel of Quality A, the value for flexural stresses being 8 tons per square inch.)

(40) The working stresses on pillars and other compression members of Quality A steel due to all loads and forces other than wind pressure shall not exceed those specified in the following table :—

Ratio of effective pillar length to least radius of gyration.	Working stresses in tons per sq. in. of gross section.	Ratio of effective pillar length to least radius of gyration.	Working stresses in tons per sq. in. of gross section.
20	7·2	140	2·3
30	6·9	150	2·0
40	6·6	160	1·8
50	6·3	170	1·6
60	5·9	180	1·5
70	5·4	190	1·3
80	4·9	200	1·2
90	4·3	210	1·1
100	3·8	220	1·0
110	3·3	230	0·9
120	2·9	240	0·9
130	2·6	—	—

Intermediate values may be obtained by interpolation.

(41) The effective pillar length to be assumed in determining the working stresses for pillars of one storey height shall be the actual pillar length.

(42) The effective pillar length to be assumed in determining the working stresses for the topmost and lowest lengths of a pillar which is continuous through two or more storeys shall be measured from floor level to floor level.

The effective pillar length to be assumed for any intermediate length of such a pillar may be taken at 0·70 of the floor to floor length for any pillar which is effectually restrained at both its upper and lower ends. For any intermediate length of such a pillar in which the restraint at one or both ends is not entirely effectual, an intermediate value between 0·70 and 1·00 shall be assessed, depending upon the degree of actual restraint.

(43) In all cases of eccentric loading on pillars, the bending moment about each principal axis is to be calculated and the resulting bending stresses added to the axial stress. The permissible stress may then be increased for cases where the ratio of effective pillar length to least radius of gyration is less than 150 beyond the figure specified in Clause (40) up to a limit of $\left[C_d + 7{\cdot}2\left(1 - \frac{C_d}{C_s}\right)\right]$, where C_s denotes the stress specified in Clause (40), and C_d denotes the compressive stress due to direct or axial load.

(44) In cases where a beam is connected to a continuous pillar, the bending moment in the pillar due to the eccentricity of the reaction from the girder may be regarded as divided between the pillar lengths above and below the level of the girder in direct proportion to the stiffnesses (moments of inertia/length) of the upper and lower lengths.

(45) In continuous pillars all bending moments due to eccentricities of loading at any one floor level may be considered as entirely dissipated at the levels of the floor beams immediately above and below, provided that the pillar at these latter levels is effectually restrained in the direction of the eccentricity.

These clauses do not set out the actual method to be used in proportioning the members of the frame, which consists, in the main, of lines of horizontal beams connected to vertical stanchions by brackets attached to the top and bottom flanges of the beams, but there is no doubt of the method implied. When considering the effect of the vertical loads arising from the dead and superimposed loads applied to the structure, it is usual to assume that the beam is simply supported, or connected to the stanchion by perfectly free hinges which apply no bending restraint. This assumption makes the choice of the necessary beam-sections easy, the vertical loads being known. Arising from this, since it is assumed that the ends of the beams are attached to the vertical stanchions by hinges, the only load coming on to a stanchion from a beam is taken to be a vertical reaction. When the beam is attached to the flange of a stanchion of I-section this reaction must lie some distance from the centre-line; conservative engineers take it to act at a distance of 2 inches outside the face of the stanchion, representing approximately the position of the centre of the bottom bracket on which the beam rests. The stanchion is designed by means of Clauses 40–45 to carry the eccentric reactions applied to it.

The frame, as Clause 35 (*f*) indicates, may be called upon to withstand other than vertical loads. Horizontal loads may be applied due to wind-action. The stresses brought about by a horizontal load acting on the frame with hinged joints can be found without difficulty, but they will be excessive, since the stanchions merely behave as cantilevers of the same length as the height of the building. Practical experience has shown the designer that the actual stresses are not excessive, so that, when considering the problem of the effect of horizontal wind-loads, it is customary to consider the connections between beams and stanchions as perfectly

rigid joints capable of transmitting any bending moment, rather than as perfect hinges. The fundamental assumptions in use to-day are, therefore, diametrically opposite, and depend on the type of load which is being considered. With these assumptions the detailed design of the beams is straightforward; that of the compression members is, however, a more difficult problem, and in considering it Clauses 40–47 set out above should be examined.

The permissible loads per unit area (working-stresses) on pillars having various slenderness-ratios are set out in Clause 40. These loads have been deduced from the well-known Perry [1] formula for the maximum stress in a pin-ended strut having an initial curvature representing the imperfections to be expected in a practical member. The factor defining curvature used is that suggested by Professor Andrew Robertson.[2] The formula is

$$Ap = \frac{p_y + (\eta + 1)p_e}{2} - \sqrt{\left\{\frac{p_y + (\eta + 1)p_e}{2}\right\}^2 - p_y p_e},$$

where p denotes the working-stress in tons per square inch of gross section.

A is a constant, taken as 2·36.

p_y denotes the yield-stress = 18 tons per square inch.

p_e „ Eulerian load in tons per square inch of gross section.

η „ 0·003 L/r.

L/r „ (length)/(radius of gyration).

It will be seen that a load-factor of 2·36 is included; that is to say, if a pin-ended strut is designed by this formula the applied load could be increased to 2·36 times the value assumed for it in design before the maximum stress in the member reached 18 tons per square inch.

The loads set out in Clause 40 are based, as has been said, on the consideration of a pin-ended compression-member. The stanchion lengths in a steel frame are not hinged at their ends but are continuous through many floors, and in an attempt to allow for this continuity Clauses 41 and 42, dealing with the effective length to be assumed, have been introduced. If a compression-member is continuous through a number of storeys and a truly axial load is applied, then, as the member deflects under this load, restraining moments are induced at the ends of each storey-length, due to the beams which

[1] W. E. Ayrton and J. Perry, "On Struts," *The Engineer*, vol. lxii (1886), pp. 464, 513.

[2] A. Robertson, "The Strength of Struts," Selected Engineering Paper No. 28, Inst. C.E.

frame into the stanchion at the floor-levels offering resistance to the change of slope of the stanchion at those points. The presence of this restraining moment means that a member of given section can safely carry a greater axial load than it would have been able to do had it been pin-ended and therefore without the restraining moments at the ends. It was thought that the simplest way for the designer to take into account the restraints provided was to assume that the effective length of the member was less than its actual length, and then to design, by means of Clause 40, the pin-ended member having a length equal to the effective pillar-length assumed. Clauses 41 and 42 were drafted before any reliable information of the effective length of pillars in steel frames was available, and they do not help the designer as no definition is given of " effectual restraint." In applying this method of design it appears to be common to-day to estimate as follows the ratio of effective to actual length for intermediate stanchion-lengths :—

Where three or four beams frame into the stanchion at each floor	0·750
Where two beams frame into the stanchion at each floor	0·875
Where one beam frames into the stanchion at each floor	1·000

For top lengths under the same conditions, the figures are 0·875, 1·000 and 1·250 respectively. Although these values are probably on the safe side for most frames they are, at best, rough approximations, since the number of beams alone cannot govern the restraint.

The magnitudes of the restraining moments at the ends of a stanchion-length depend not only on the beam-stiffness but also on the type of connection between the beams and the stanchion. If the beams are connected to the stanchion by perfectly free hinges, then, no matter how heavy the section of the beam, no restraining moments can be passed from the beams to the stanchion, and therefore the effective length of the stanchion is not decreased by the presence of the beams. If, on the other hand, the connections between the members have considerable rigidity then appreciable restraining moments are brought into play, and it has been found possible [1] to estimate the value of the ratio of effective length to actual pillar-length for a member in a frame with rigid joints as shown in Table I.

[1] J. F. Baker, "Note on the Effective Length of a Pillar Forming Part of a Continuous Member in a Building Frame," Second Report of the Steel Structures Research Committee (1934), p. 13.

TABLE I.—SUGGESTED VALUES OF THE RATIO OF EFFECTIVE LENGTH TO ACTUAL PILLAR-LENGTH FOR A MEMBER IN A FRAME WITH RIGID JOINTS.

$\dfrac{\text{Stiffness of beam}}{\text{Stiffness of pillar-length}}$ . .	0·25	0·50	1·00	1·50 and above
$\dfrac{\text{Effective length}}{\text{Actual pillar-length}}$ . . .	0·90	0·75	0·65	0·60

Where the usual types of semi-rigid steelwork connections are used the variables are so many that it is impossible to formulate a concise rule. The designer is forced, therefore, to depend on the unsatisfactory approximations already mentioned.

Clauses 43, 44 and 45 deal with the effect of the bending moment induced by the reactions coming from the beams. If the loads on the beams at a floor-level produce reactions R_1 and R_2 on either side of the stanchion at eccentricities e_1 and e_2 (usually taken as half the width of the stanchion plus 2 inches), then the total moment applied to the stanchion at that floor-level is calculated as $R_1e_1 - R_2e_2$, and, according to Clause 45, no account is taken of the effect on this moment of loads on the floors above and below. The end bending moments in the stanchion-lengths above and below the floor-level under consideration are found, according to Clause 44, by dividing the total moment between them in the ratio of their stiffnesses. The end bending moment applied to a stanchion-length produces bending stresses in the member, and the designer must decide whether they can be sustained by the member. The working-stresses of Clause 40 were deduced from a consideration of a pillar subjected to axial load only, so that some alteration is called for when an end moment also is present. The alteration is made by means of the formula in Clause 43.

This formula is open to severe criticism, but it is not easy to set it down without discussing the whole problem of the behaviour of compression-members. The total maximum stress in a pin-ended pillar of cross-sectional area A, carrying an axial load P, is the sum of the unit compressive stress P/A and the maximum bending stress developed by a bending moment Py, where y is the greatest deflection of the pillar due to the action of P. The working-stress of Clause 40 gives the value of P, expressed as P/A, which makes the maximum stress in the member rise to a certain limiting value. The reasoning behind the formula of Clause 43 is that if an axial load P', less than P, acts together with an end bending moment, then, since the greatest deflection of the strut due to the action of P' alone is less than y, the end bending moment may develop a bending

stress somewhat greater than $(P-P')/A$ without the maximum stress in the member rising above its limiting value. The fault in this reasoning is that the effect of the end bending moment on the magnitude of y is neglected. As will be seen later, when the end bending moment produces "single curvature" bending (p. 181) y is increased, so that Clause 43 overestimates the permissible stress. There are other objections to the formula of Clause 43, the only virtue of which is its simplicity, but enough has been said to show that there is evidence of considerable confusion of thought when dealing with compression-members.

One other point needs emphasizing. In the present method of design it is assumed, when estimating the stresses in members, that all members are loaded. This may not produce the most rigorous stress-conditions. For instance, in calculating the bending moment on a stanchion a greater moment (R_1e_1) is produced when the beam on one side only is loaded than that $(R_1e_1 - R_2e_2)$ when both are loaded.

The designer's task is by no means an easy one, and simple formulas must be derived for his use; it is evident, however, from a study of existing design-methods, that there has been a tendency in the past to base the formulas on assumptions which make some calculation possible rather than on those justified by the actual behaviour of the frame.

Preliminary Investigations.

Experimental Frame.

The use of the design-method reviewed above results in the production of safe structures. This is due, in the main, to the assumption of values for the superimposed loads which are greater than those actually applied to the structure. It was clear to those who had taken part in the drafting of the recommendations for a Code of Practice that a limit had been reached in the reduction of these values, below which it would be unsafe to venture while the present somewhat unsatisfactory methods of estimating the stresses induced in the structure were in use. It was equally clear that, as the present method does produce safe structures, there were opportunities of making further economies in material. These could only be exploited if a method of design having a rational basis could be produced. The production of such a method was the main problem facing the Steel Structures Research Committee.

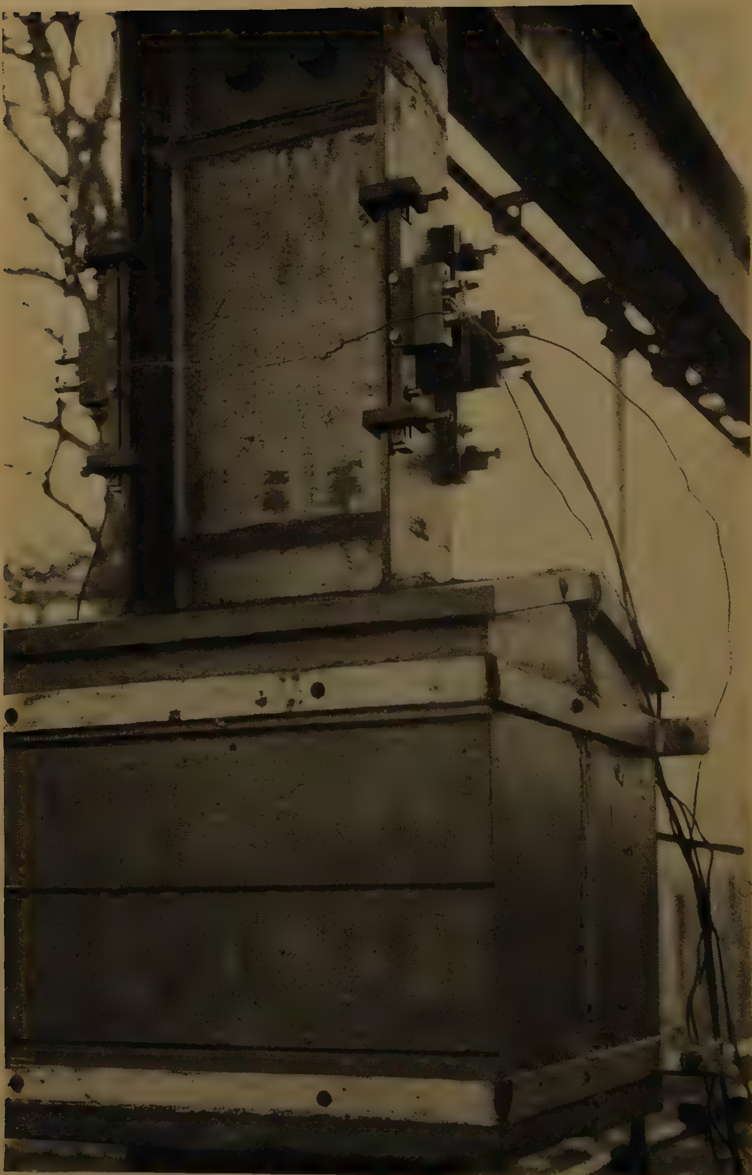
Although, strange as it may seem, there was no information of any value available as to the real behaviour of framed structures, the contradictory assumptions made in design had drawn attention

B

Fig. 1.

GENERAL VIEW OF EXPERIMENTAL FRAME.

Fig. 2.



MAIHAK GAUGES ATTACHED TO A STANCHION.

to the beam-to-stanchion connections. One of the first tasks put in hand, therefore, was a laboratory investigation of their behaviour. This was undertaken by Professor Cyril Batho, M. Inst. C.E., and was carried out at Birmingham University. At the same time it was realized that, if a design-method was to be produced, an accurate estimate of the stress-distribution in any frame would have to be possible, and to this end the production of a sound method of stress-analysis was essential.

It would have been of little value to embark on an elaborate mathematical investigation without the aid of experiment, and it was decided, therefore, to erect a steel framework, the stress-distribution in which could be measured and which would serve to guide the mathematical analysis.

The frame, which is shown in *Fig. 1*, was erected at the Building Research Station at Watford. It consisted of six stanchions, the bases of which were bolted down to a concrete raft, and twenty-one beams, all 8-inch by 4-inch by 18-lb. steel joists. Provision was made for the use of various types of bolted and riveted beam-to-stanchion connections. Three of the stanchions were arranged with their webs parallel to the length of the frame and three with their webs perpendicular to the length of the frame, so that a large variety of one- and two-bay three-storey frames could be built up. The cylinders shown hanging from the beams were provided for loading-purposes. A reservoir was built on a neighbouring site and through a system of pipes it was possible to fill with water any or all of the tanks. Unfortunately, the time taken in loading and unloading by this method was so great that in most of the tests carried out a less elaborate scheme was followed, and a central concentrated load was applied to each beam by means of a spring and turnbuckle. The object of the tests on this frame was the determination of the stress-distribution in the members due to the application of external loads. The stress-distribution was found by measuring the strains at a number of sections of the members. Many different types of strain-gauge were tested, and it was found that most of them were quite unsuitable for use in the open. In view of the very large number of observations which the complete programme of tests involved, the only possible instrument was one which could be protected and left undisturbed when readings were taken, so enabling the work to be carried on in any weather.

The Maihak extensometer was finally selected. The instrument consists of a central receiver and a number of gauges, some of which are shown, in *Fig. 2*, attached to a stanchion. Below them will be seen a " beehive " type of box, which could be placed at any section of the frame and thus protected all the gauges there from the weather.

The essential part of each gauge is a 12-centimetre length of high-tensile steel wire, stretched between two knife-edges which are clamped to the member under test. A small electro-magnet, fixed above the gauge-wire, is connected to the central receiver. By pressing a morse-key in the receiver the gauge-wire is caused to vibrate and the note is heard in a pair of headphones. A standard wire, similar to the gauge-wire, is contained in the receiver; it is fitted with a micrometer adjustment by means of which the tension in the wire is controlled. The standard wire also can be set into vibration and the note from it heard superimposed on the note from the gauge-wire. By adjusting the tension in the standard wire until the two notes are the same, a measure of the tension in the gauge-wire is obtained from the reading of the graduated head of the micrometer adjustment. The strain in the member under test is deduced from the difference between the readings taken before and after loading. It was normal practice to attach four gauges to the edges of a member, as shown in *Fig. 2*, the mid-points of the test-wires lying in the plane containing the section of the member at which the stress-distribution was required. Two small clamps were used to fix each gauge in that position on the face of the flange, close to the edge, which preliminary trial had shown to be the most satisfactory even when the gradient of stress across the flange was steep. The axial stress and the bending stresses about both principal axes of the member were deduced from the strain-readings at the four gauge positions, on the usual assumption that plane cross sections of the member remained plane after bending.

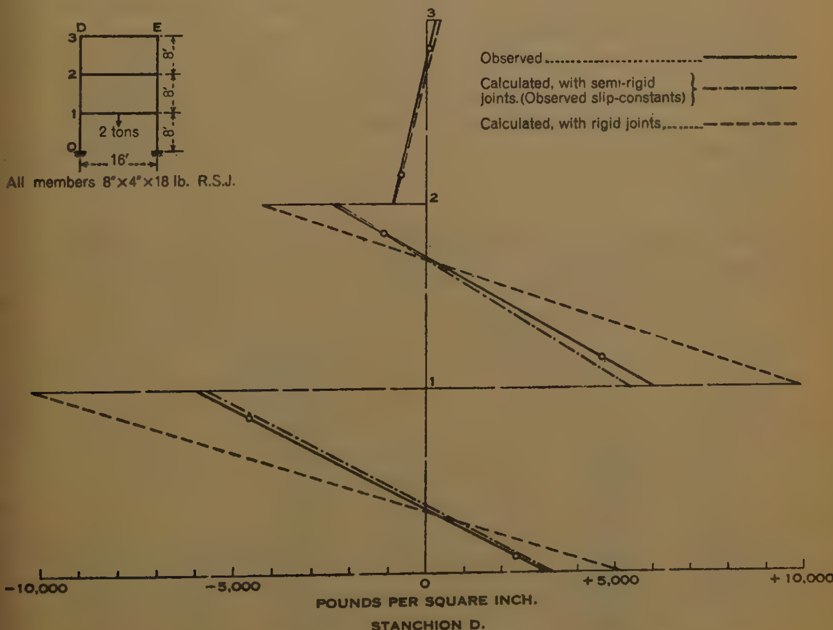
One of the first tests carried out was on the symmetrical single-bay frame shown in *Figs. 3*. The beams were connected to the webs of the stanchions by type-"A" connections consisting of top and bottom flange-cleats made up of $3\frac{1}{2}$-inch by 3-inch by $\frac{5}{16}$-inch angle, 4 inches long, and secured with $\frac{1}{2}$-inch diameter bolts. The distribution of observed stresses when a central concentrated load of 2 tons was applied to the beam at the first floor is shown in *Fig. 3* and in Table II. In the Table are set out the maximum stresses (+ denoting tension, — compression at the inside edge of the member) at the top and bottom of each stanchion-length due to bending about the YY axis of the stanchion. The stresses in the stanchion for this condition of loading have been calculated on the assumption that the joints are perfectly rigid. They are shown in *Figs. 3* and Table II (col. 4). The fact is revealed that, although the bending stresses actually induced in the stanchion-lengths are smaller than those which would have been found had the joints been perfectly rigid, the distributions of stress in the two cases are exactly similar in form, with the result that appreciable

TABLE II.—COMPARISON OF BENDING STRESSES. SYMMETRICAL SINGLE-BAY FRAME, FITTED WITH TYPE-"A" CONNECTIONS. CONCENTRATED LOAD OF 2 TONS AT CENTRE OF BEAM D_1E_1.

Section.	Observed, with type-"A" connections: lbs. per square inch.	Calculated, with semi-rigid joints (observed slip-constants): lbs. per square inch.	Calculated, with rigid joints: lbs. per square inch.
D_0D_1	+3,100	+3,345	+ 5,176
D_1D_0	−6,075	−5,775	−10,352
D_1D_2	+5,800	+5,354	+ 9,916
D_2D_1	−2,450	−2,364	− 4,302
D_2D_3	− 850	− 890	− 831
D_3D_2	+ 200	+ 244	+ 350

bending stresses are found in the top length 2—3 of the stanchion which is two storeys above the applied load. The magnitude of the

Figs. 3.



MAXIMUM FIBRE-STRESSES DUE TO BENDING IN PLANE OF FRAME.
LOAD: 2 TONS AT CENTRE OF D_1E_1.

observed bending stress is considerable. It must be remembered that the beam-to-stanchion connections were not of a particularly

rigid type, and would in any case have been assumed in the present design-method to have been pin-ended so far as vertical loads on the beam were concerned. As the connection was to the web of the stanchion the eccentricity would have been taken by many designers as zero, and by the conservative designer as not more than 2 inches. In actual fact the "equivalent eccentricity" of the connection to stanchion D at the level of the first floor (that is to say, the distance from the axis of the stanchion at which the reaction arising from a similarly-loaded simply-supported beam would have had to act to produce the observed bending stresses in the stanchion), was 9·3 inches. The form of the bending-stress or bending-moment diagrams observed for these stanchion-lengths are typical for a frame in which sway, that is, relative horizontal deflection of the beams in the plane of the frame, does not exist. The condition of bending in such stanchion-lengths may be defined as "double-curvature" bending.

A different condition is found when sway is present, and this can be most easily seen from the results of tests on an unsymmetrical frame, *Figs. 4*, in which the beams were joined by type-"A" connections to the flange of one stanchion D and to the web of another stanchion A. The stresses observed in stanchions D and A due to the application of a central concentrated load of 2 tons to the beam at the first floor are shown in *Figs. 4* and are set out in Table III.

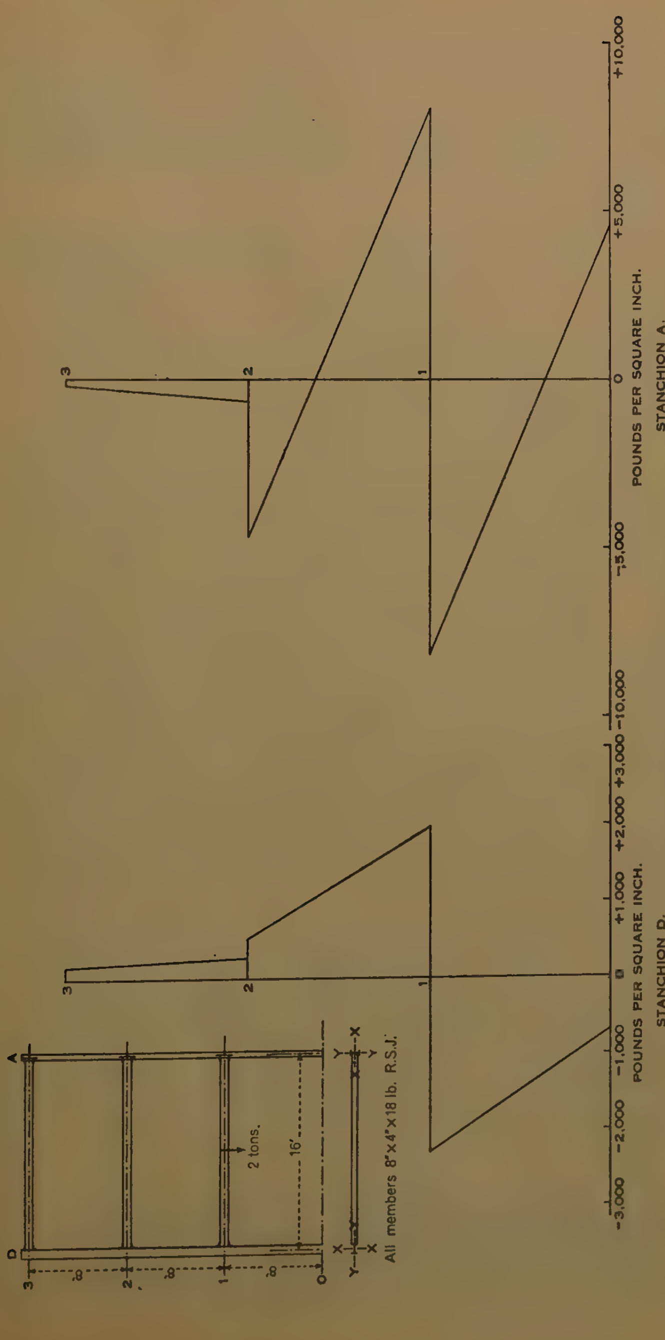
TABLE III.—OBSERVED BENDING STRESSES. UNSYMMETRICAL SINGLE-BAY FRAME FITTED WITH CONNECTIONS OF TYPE "A."

CONCENTRATED LOAD OF 2 TONS AT CENTRE OF BEAM D_1A_1.

Section.	Observed, with type-"A" connections: lbs. per square inch.
A_0A_1	+4,560
A_1A_0	−8,120
A_1A_2	+8,100
A_2A_1	−4,520
A_2A_3	− 600
A_3A_2	− 200
D_0D_1	− 700
D_1D_0	−2,275
D_1D_2	+2,000
D_2D_1	+ 525
D_2D_3	+ 250
D_3D_2	+ 150

It will be seen that the forms of the bending-stress diagrams in the two stanchions are quite different, owing to the sway which has arisen

Figs. 4.



Maximum Fibre-Stresses due to Bending in Plane of Frame.

from the lack of symmetry in the structure. In the lengths 0—1 and 1—2 of stanchion A, "double-curvature" bending has resulted as before from the application of the load to the beam, whereas in stanchion D the form of bending can be designated "single-curvature" bending. The difference in these forms was found to be of the first importance when the rational design of a stanchion-length was under consideration.

In addition to measurements of strains in these frames the characteristics of the beam-to-stanchion connections were found. Relative rotation of the ends of the members joined was measured by a simple arrangement of dial-indicators, and it was possible to plot a curve showing the relations between the moment transmitted by a connection and the relative rotation of the members joined. It was found that the relation between moment and relative rotation (the characteristic curve for the connection) was by no means linear, and that on the removal of load from a beam reverse bending moments remained at the ends of the beam. These points will not be discussed in detail here, as the same behaviour was found in the laboratory tests on connections and in tests on existing buildings, which will be discussed at greater length at a later stage in the Paper. They had their importance at the time, since the work on the experimental frame was designed to guide the production of a method of stress-analysis that would enable an accurate estimate of the stresses to be made in any frame fitted with any form of steelwork connection. It was found possible, making the assumption that the relation between moment and rotation was linear, to produce a fairly simple method of analysis, so that, by representing the characteristic curves for the connections by straight-line chords, the stresses in any frame fitted with semi-rigid connections could be estimated accurately. An example of the stresses calculated in this way is shown in Table II (Col. 3). Little would be gained in describing the method of analysis in any detail. One method, based on the well-known slope—deflection equations, together with a mechanical method of analysis which proved to be of the greatest assistance in the early work, has already been described in a Paper published by The Institution.[1] Subsequently it was found possible to extend the moment-distribution method of Professor Hardy Cross [2] to take into account the effect of semi-rigid connections. All these methods were essential in the examination of the large

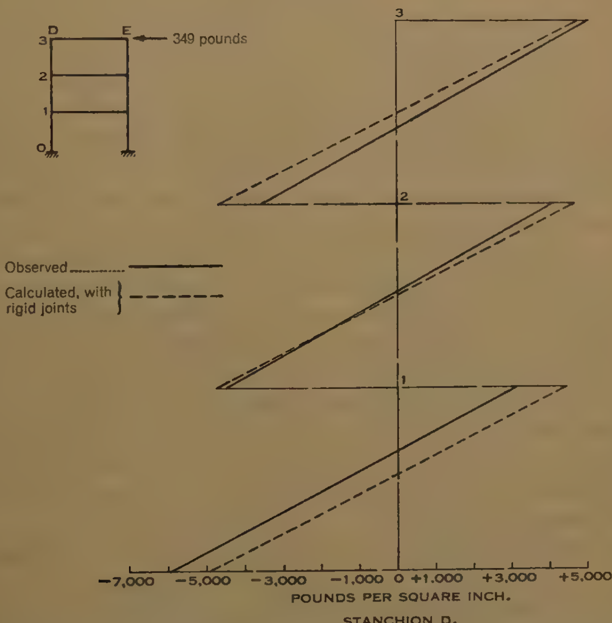
[1] J. F. Baker, "The Mechanical and Mathematical Stress-Analysis of Steel Building-Frames," Selected Engineering Paper No. 131, Inst. C.E.

[2] H. Cross, "Analysis of Continuous Frames by Distributing Fixed-End Moments," Trans. Am. Soc. C.E., vol. 96 (1932), p. 1.

number of frames of different types which had to be studied before a method of design could be proposed.

One other test carried out on the experimental frame must be mentioned here. A horizontal load of 349 lbs. was applied to the top of the symmetrical single-bay frame (*Figs. 3*). The observed stresses are plotted in *Figs. 5*, and are set out in Table IV. It will

Figs. 5.



Maximum Fibre-Stresses due to Bending in Plane of Frame.
Horizontal Load of 349 pounds.

Table IV.—Comparison of Bending Stresses due to a Horizontal Pull of 349 lbs.

Symmetrical Single-Bay Frame.

Section.	Observed stress: lbs. per square inch.	Calculated stress, with rigid joints: lbs. per square inch.
D_0D_1	−5,700	−4,990
D_1D_0	+3,600	+4,590
D_1D_2	−4,600	−4,780
D_2D_1	+5,050	+4,780
D_2D_3	−4,250	−4,690
D_3D_2	+5,150	+4,880

be seen that, while the distribution of stress was of the same form, the maximum bending stress observed at the foot of each stanchion was greater than would have been predicted on the assumption of perfectly rigid joints.

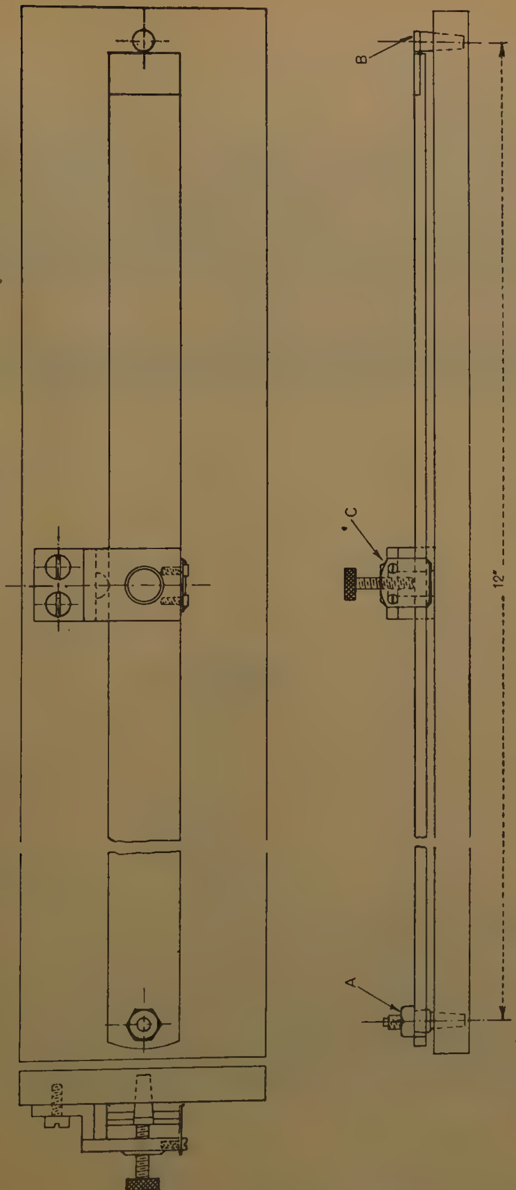
New Geological Museum.

While the work on the experimental frame was of the greatest value, it was realized that different conditions might exist in actual full-scale frames, and a search was made for a building which could be tested. The Committee was most fortunate in having the steel framework of the Museum of Practical Geology in South Kensington placed at its disposal by H.M. Office of Works. Testing an existing building is no easy matter, and the Committee was fortunate that, in its first attempt in work of this kind, access was obtained to a framework which was completed before any clothing was added. This is a state of affairs rarely found, since the great attraction of the steel-frame building is its rapidity of erection, so that, as a rule, the steelwork advances only a little ahead of the laying of the floors and the building of the walls and fire-resisting casing. One or two attempts had been made in other parts of the world to discover the behaviour of a steel frame by making strain-measurements on existing structures, but the results had always been inconclusive, the instruments used being unfitted for the work. It was realized that, for the Committee's tests, special instruments would have to be designed.

The conditions which are experienced on an actual building-frame make it essential that the strain-gauge and any magnifying device incorporated in it should be robust and capable of withstanding any shocks transmitted to it from the building operations in progress during the test-period. It was clear, therefore, that the instrument to be used on the Geological Museum must be as simple and robust as possible, and, as no time was available in which to carry out a protracted test of the reliability of instruments, it was decided that any magnifying device which might be sensitive should be kept distinct from that part of the strain-gauge which was to be permanently attached to the members.

The part of the strain-gauge which was attached to the member is shown in *Figs. 6.* Two tapered pins A and B were driven into the member 12 inches apart. A carried a mild-steel bar, the free end of which was separated from B by a gap of 0·008 inch. Any relative movement of the two pins due to the strain in the member over the 12-inch gauge-length was the same as the relative movement of the free end of the bar and the pin B. This movement could be

Figs. 6.



STRAIN-GAUGE.

conveniently measured by means of a micrometer microscope, which was only brought into position when a reading was required.

The chief difficulty in this method was to obtain reliable marks

on the end of the bar and on the pin B, the distance apart of which was to be measured. Satisfactory results were obtained by soldering a strip of unworked stainless steel to the free end of the bar and a top of similar steel to the pin B. The stainless steel surfaces were then carefully prepared on a fine emery-paper and an indentation was made on each with a diamond similar to that used in the Vicker's diamond hardness-test. The marks thus made appeared under the microscope as black squares and the distance between their corners, which were sharp and undistorted, could be read accurately. A bridge-piece C (*Figs. 6*) was also attached to the member, and served two purposes. The spring forced the bar against a half-round stop soldered inside the bridge-piece and thus prevented the bar rotating about A, whilst leaving it free to move longitudinally, while the milled-headed screw in the top of the bridge-piece enabled the bar to be depressed slightly so as to bring the surface of the stainless-steel tip to the same level as the surface of the pin. The strain-gauge, when erected on the member, was protected by a cast-iron bottomless box, 16 inches long, 3 inches wide and $2\frac{3}{4}$ inches deep, attached to the member by three lugs, D, E, and F, *Fig. 7*. A sheet of asbestos was fastened inside the lid to prevent as far as possible the contents of the box being affected by purely local temperature-conditions.

The microscope used for measuring the distance between the indentations is shown (*Fig. 8*) strapped to a girder. The micrometer eyepiece was fitted with a traversing knife-edged shutter, in addition to the usual cross-hair, and a vertical illuminator was provided. The microscope-tube was attached to a plate N which slid on the surface of an aluminium casting, P, giving a slow motion in all directions. The aluminium casting stood on three legs and was fixed in any position by a strap passing around the member. A right-angled eyepiece was provided which could be fitted to the microscope for reading certain gauges in awkward positions.

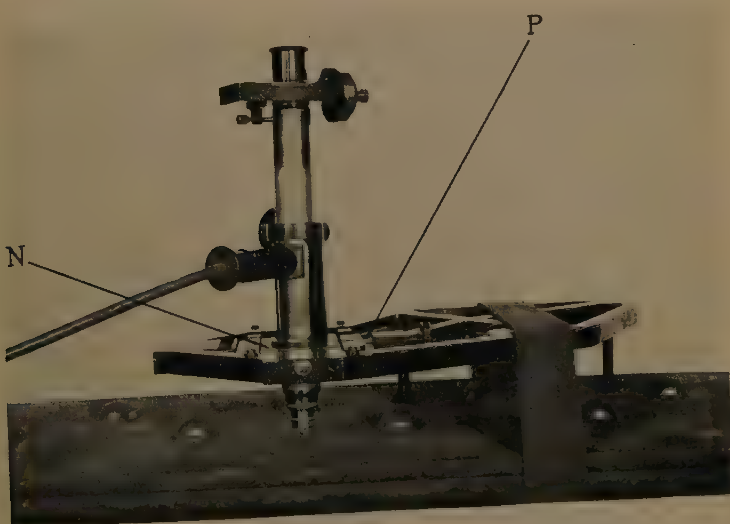
The micrometer-head was divided into one hundred divisions. By turning it through one division the shutter was moved a distance corresponding to approximately 0·00004 inch on the gauge. With a gauge-length of 12 inches, therefore, on a structural-steel member, a relative movement of the indentations measured by one division of the micrometer-head corresponded to a stress in the member of about 100 lbs. per square inch. The preliminary investigation carried out with this instrument showed that in the hands of a skilled observer the readings could be relied upon to at least ± 200 lbs. per square inch, these limits including any variation due to temperature-changes of the order likely to be experienced during the tests.

The frame of the Geological Museum was of five storeys, part of it consisting of a series of galleries around a central well. Attention

Fig. 7.

CAST-IRON BOX WITH LID REMOVED, SHOWING STRAIN-GAUGE IN PLACE.

Fig. 8.



MICROMETER MICROSCOPE USED FOR MEASURING THE DISTANCE BETWEEN INDENTATIONS.

B

was concentrated on one of the inside gallery-stanchions forming the well, and on the beams connected to it. The shaft of the gallery-stanchion was made up of two 14-inch by 6-inch by 57-lb. I-sections with cover-plates in the bottom stanchion-length 16 inches by $1\frac{3}{4}$ inch, and in the storey-length above, 16 inches by $1\frac{1}{2}$ inch, 16 inches by 1 inch and 14 inches by $\frac{1}{2}$ inch, respectively. Each beam consisted of two 24-inch by $7\frac{1}{2}$-inch by 100-lb. I-sections with 16-inch by $1\frac{1}{4}$-inch flange-plates on the ground-floor beam and 16-inch by $\frac{5}{8}$-inch flange-plates on the first-, second- and third-floor beams. Each beam-to-stanchion connection consisted of a stiffened bottom bracket and a top cleat made of 4-inch by 4-inch by $\frac{1}{2}$-inch angle, the vertical leg of which was connected to the stanchion by three $\frac{7}{8}$-inch-diameter rivets.

Sixty-four gauges were attached at sixteen sections of the stanchions and beams. The work was done under very comfortable working conditions, as a tower of scaffolding was built around the stanchion with platforms at each level where gauges were to be placed. The measurements of the strain when load was applied to the framework, and the reduction of the observations, were, for this particular building, in the hands of Dr. Oscar Faber, M. Inst. C.E., and a full report of the work has been made elsewhere.[1] A distributed load equal to that assumed in the design-calculations was applied to each beam of the bare frame in turn, and strain-measurements were made before and after each load was applied. The behaviour of the stanchion was, in general, similar to that found in the experimental frame. Each stanchion-length bent in "double curvature" and the bending stresses were greater than those which are usually assumed in design. The "equivalent eccentricity" of each connection was approximately twice that which would have resulted from the usual assumption that the beam-reaction acts at the centre of the cleat, in this case 2 inches outside the face of the stanchion. The tests were repeated after the fire-resisting casing and the floor-slabs were in position. The casing was heavy and consisted of a reinforced-concrete cover to the stanchions, outside dimensions being $23\frac{1}{2}$ inches by $23\frac{1}{2}$ inches; the width of the casing on the beams was $23\frac{1}{2}$ inches and the floors were of the filler-joist type having a thickness of 6 inches. The striking deduction drawn from the results of the tests on the cased frame was that the effect of the casing, as far as the stanchion bending stresses were concerned, was negligible. An examination of the observed stresses

1 "Observed Stresses in a Steel-Frame Structure at the Museum of Practical Geology, South Kensington," Second Report of the Steel Structures Research Committee (1934), p. 44.

at the mid-span of the beams appeared to indicate that the floor-slabs did assist in carrying some of the compression which would otherwise have been carried by the top flange of the steel girders, but this did not seem to have any influence on the moments at the ends of the beam or in the stanchions. While the beam-to-stanchion connections in this frame were light compared with the sections of the members, the casing and the floors were very much heavier than those normally found in steel-framed office or residential buildings. This made it desirable to carry out further tests on more representative frames, since it was realized that, before a method of design could be recommended, adequate proof would be needed that it gave a true representation of the stresses in the steel framework when it formed part of the complex structure consisting of floors, walls and fire-resisting casing which goes to make up the finished building.

Further Tests on Existing Buildings.

It was too much to hope for a second opportunity similar to that given to the Committee by H.M. Office of Works, and attention was concentrated on the development of a technique of testing which could be applied to the framework of a building in course of construction. Whilst, from most points of view, it is desirable to apply a distributed load to the structure tested, this proved so costly in the case of the tests on the Geological Museum that arrangements had to be made to use a concentrated load. The experience so far obtained showed that, if all the points which were in doubt were to be cleared up, strain-readings would be needed at many more sections of the steelwork than the sixteen on the Geological Museum, and this, coupled with the fact that the sixty-four gauges on that frame took 3 months to fit, made it evident that the type of gauge used there, satisfactory though it was, could not be made use of in tests on a building erected under normal commercial conditions. Further experience gained on the experimental frame showed that the Maihak gauge used there could be satisfactorily adapted for tests on existing buildings.

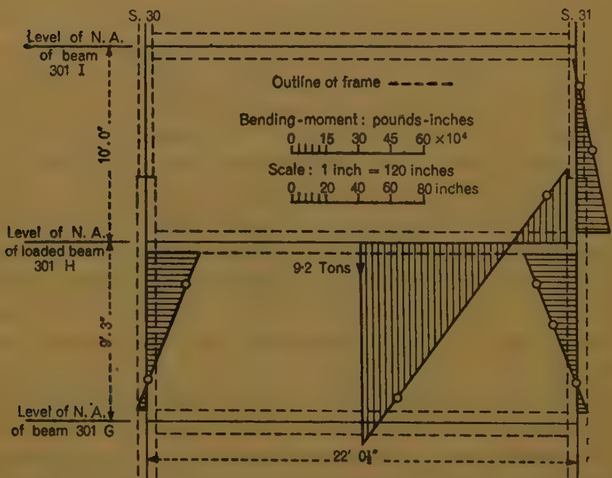
Early in 1933, by the courtesy of Messrs. Dorman, Long & Company, access was obtained to an hotel building which was then in the early stages of construction. Tests were carried out on two parts of the frame. In one, load was applied to the beams of a symmetrical single-bay frame, and in the other to beams which framed into an internal stanchion at one end and into the middle of a wall-beam at the other. The conditions under which the tests were carried out were very different from those experienced at the Geological Museum. As the structure was being erected under

ordinary commercial conditions nothing could be attempted which would hold up the normal progress of the building. The speed of erection (normally only 3 days elapsed between the laying of one floor and the next above), played a big part in determining the details of the programme and of the apparatus used. Thirty-two Maihak gauges were available for this work and strain-readings could, therefore, be taken at only that number of points in any one test. In the interests of economy and mobility the minimum of scaffold was used on the bare frame, and it was, therefore, at times impossible to have more than one investigator attaching the gauges at a section. Some protection against rain and dust had to be provided for the instruments, and it was decided to use a light five-sided box covering each gauge, instead of the heavy beehive-type shown in *Fig. 2*. Heavy clamps were used for attaching the gauges to the members, as it was realized that they might be subjected to more severe vibrations than those encountered on the experimental frame. No loosening of clamps or slipping of the gauges occurred, except when riveting with a pneumatic tool was carried out less than 15 inches from a set of gauges. The insertion of rivets into a stanchion at a height of one storey above the gauges did not affect the clamps. Riveting anywhere on the frame did, however, interfere with the work, as a comparatively slight vibration was sufficient to cause painful humming in the amplifiers which formed part of the central receiver, and to make accurate reading difficult. Riveting was not the only cause of this trouble. The carpenters' hammers were just as annoying when forms were being put together near the receiver.

The short time available for each test influenced the choice of loading tackle as well as that of the strain gauge. It was decided to apply one point-load to each beam under test. A shackle, from which was suspended a link carrying a snatch-block, was placed around the beam to be loaded. A wire rope having one end fixed to the concrete raft forming the foundation of the building, and having the other attached by straining screws to three crane-blocks resting on the raft, passed through holes in the floors and over the snatch-block. By turning the straining screws the crane-blocks, weighing approximately 1 ton 18 cwt. each, could be lifted from the ground. Additional load was obtained by adding twenty 1-cwt. weights to the crane-blocks. In this way a concentrated load of nearly 14 tons could be applied to a beam in four increments. The link between the shackle and snatch-block, to which two Landon extensometers were attached, was made up of $1\frac{1}{2}$-inch by $\frac{1}{2}$-inch mild-steel plate, 24 inches long. By measuring the strains in this link an estimate of the load applied to the shackle could be made to within 200 lbs.

The first series of tests was made on the steel frame before floors, walls or stanchion-casing had been built. A concentrated load was applied to each beam in turn and strain-measurements were made, exactly as in the experimental frame, at two sections of the beam and at two or three sections of each stanchion-length. The load was applied in three or four increments, removed and reapplied a number of times. A typical bending-moment diagram deduced from the strains measured on a single-bay portion of the building is shown in *Fig. 9*. It will be seen that an appreciable restraining moment is present at the end of the beam, reducing the maximum stress in the member by 27 per cent. below the value for the similarly-loaded simply-supported beam, with corresponding moments in the

Fig. 9.



stanchion-lengths above and below. The diagrams for the stanchion-lengths are similar to those obtained on the experimental frame when sway was absent, each stanchion-length bending in "double curvature." The distance of the point of contraflexure in the beam from the axis of the stanchion gives the "equivalent eccentricity" of the connection. In this particular case it was 39·6 inches, whereas, since the stanchion was a 12-inch by 8-inch by 65-lb. joist, the value of the eccentricity assumed in design would not have been more than 8 inches. The beam-to-stanchion connection which gave rise to this eccentricity, five times as great as is normally assumed, consisted of a bottom bracket 8 inches by 4 inches by $\frac{3}{4}$ inch, 8 inches long, a top cleat 4 inches by 4 inches by $\frac{1}{2}$ inch, 8 inches long, and web-cleats formed of two angles 4 inches by 4 inches by

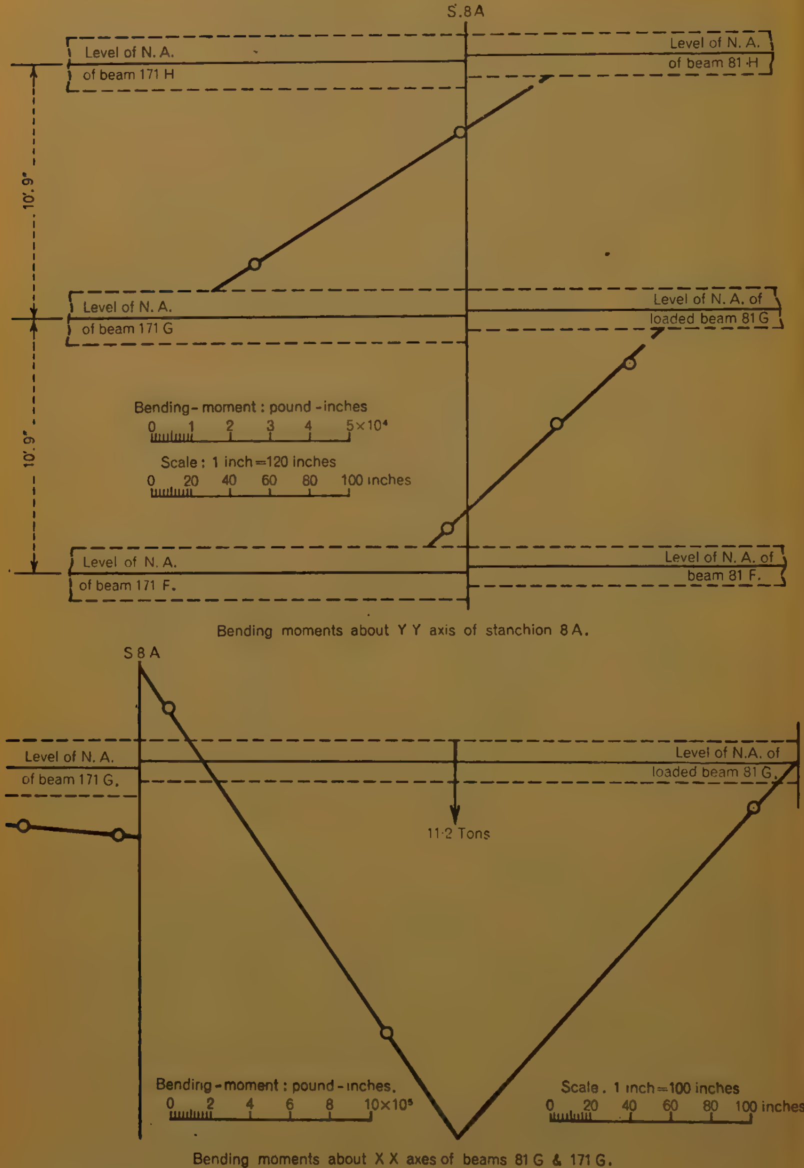
$\frac{1}{2}$ inch, 10 inches long, attached by $\frac{7}{8}$-inch diameter rivets to the stanchion and to the web of the beam, which was a 14-inch by 8-inch by 70-lb. joist. It will be seen that the ratio of the observed bending moments in the stanchion immediately above and below the loaded beam was 0·65, while the corresponding ratio of the stiffnesses of the stanchion-lengths, into which any moment coming from the beam would be divided according to the existing method of design (clause 44, p. 132), was 0·72.

Four other beams of the single-bay frame were loaded and the bending-moment diagrams obtained were all similar to that shown in *Fig. 9*. Further tests were carried out after hollow-tile floors had been laid and also after the walls and stanchion-casings had been built, but although the magnitudes of the stresses induced in the members were changed by this clothing, the forms of the bending-moment diagrams were as before.

A typical bending-moment diagram for an internal stanchion (No. 8A) and the beams framing into it, is shown in *Figs. 10* (p. 152). A central concentrated load was applied to a beam, which framed at one end into the web of the stanchion, the connection being of the type described above in the single-bay portion of the building, and at the other to the middle of a wall-beam. It will be seen that a considerable restraining moment was developed at the end of the beam framing into the stanchion, while at the other end no restraining moment was provided by the wall-beam. The "equivalent eccentricity" at the stanchion end of the beam was 36·6 inches, which is about 20 times more than the eccentricity which would in all probability have been assumed in design, since the connection was to the web of the stanchion. It should be noted that the beam on the far side of the stanchion received an appreciable bending moment; the possibility of this is very often forgotten in design. The form of the bending-moment diagrams for the stanchion is similar to that obtained in the single-bay portion of the building, the stanchion-lengths bending in "double curvature." One important point which should be noticed, however, is that the ratio of the observed moments in the stanchion, immediately above and below the loaded beam, was 1·34, which is considerably greater than 0·89, the stiffness of the stanchion-lengths. Two other beams in this bare frame were loaded, and the tests were repeated after floors and stanchion-casings had been built. In all cases the behaviour of the frame was similar to that shown in *Figs. 10*.

The tests on this building gave such valuable data that it was considered desirable to carry out further work on a building of a slightly different type. It will be seen that the beam-to-stanchion connections of the hotel building, formed of flange-cleats and web-

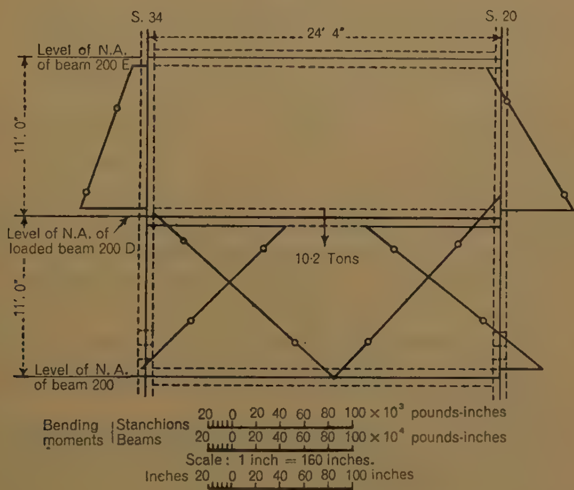
Figs. 10.



Bending moments about X X axes of beams 81 G & 171 G.

cleats, were particularly heavy. On the other hand the stanchion-casing, formed of hollow tiles, was light. It was decided, therefore, that a building having lighter connections and heavier stanchion-casing should be tested. Once again Messrs. Dorman, Long & Company were able to find the type of building required; this was an office block, and work was begun on it. The method of testing was exactly the same as that used on the hotel building except that it was found possible, through the generosity of one of the contractors, to apply distributed loads to certain of the floors. The first series of tests was made on a single-bay portion of the building which, through differences in the sections of the stanchions, was

Fig. 11.



unsymmetrical. A typical bending-moment diagram is shown in *Fig. 11.* This, as far as the beam is concerned, is similar to the diagrams shown for the hotel building in that restraining moments are present at the ends of the beams, although that at the end framing into stanchion No. 34 is relatively small. The form of the bending-moment diagrams for the stanchions is, however, somewhat different, the upper length of stanchion No. 34, for instance, showing that bending was in "single curvature." This, as will be realized if a comparison is made with *Fig. 4*, is an indication that sway had been induced in the unsymmetrical frame due to the central load on the beam. When the floors were laid in this frame and a concentrated load was applied to the beam the form of the bending-moment diagram for the stanchions showed that sway had, to a large extent,

been eliminated, and after the walls had been built it was found that, even under a distributed load applied to the bays on either side of the beam, sway in the frame was inappreciable; that is to say, the stanchions bent in "double curvature," as they had been found to do in the hotel building. This is shown clearly in *Fig. 12*, the bending-moment diagrams being those for three lengths of stanchion No. 34 (after floors had been laid and the stanchions had been cased in brickwork), due to the application of a distributed load of 13·8 tons, or approximately 88 lbs. per square foot of floor-area, to the bays of the floor on either side of beam No. 200D. This figure shows clearly that the bending of the stanchion-lengths was in "double curvature," and, what is just as important and most satisfactory, that the form of the diagram is similar to that shown for the experimental frame (*Fig. 3*) and to that given by the theoretical stress-analysis developed early in the investigation; the bending moment developed in the stanchion-length two storeys away from the applied load was appreciable.

A two-bay portion of this office building was loaded and the behaviour was much the same as that already found, the "equivalent eccentricities" of the connections (made up of flange-cleats of 6-inch by 2½-inch by ⅝-inch angle, 6 inches long, connected to the stanchion by two ⅞-inch-diameter rivets) ranging between 22·3 inches and 20·3 inches. The bending-moment diagrams showed that, although the load was applied to one beam only, producing, therefore, an unsymmetrical condition of loading, the effect of sway of the frame was inappreciable when the floors had been laid and the stanchions cased.

Further tests were carried out on the light steel framework of a block of residential flats where the observations were confined to the beams. The information obtained from these buildings was so considerable that it cannot be given in any detail here. Full accounts are to be published shortly,[1] and nothing will be attempted in this Paper except for a short summary of those points which have a direct bearing on the production of the Committee's method of design.

Review of Tests on Existing Buildings.

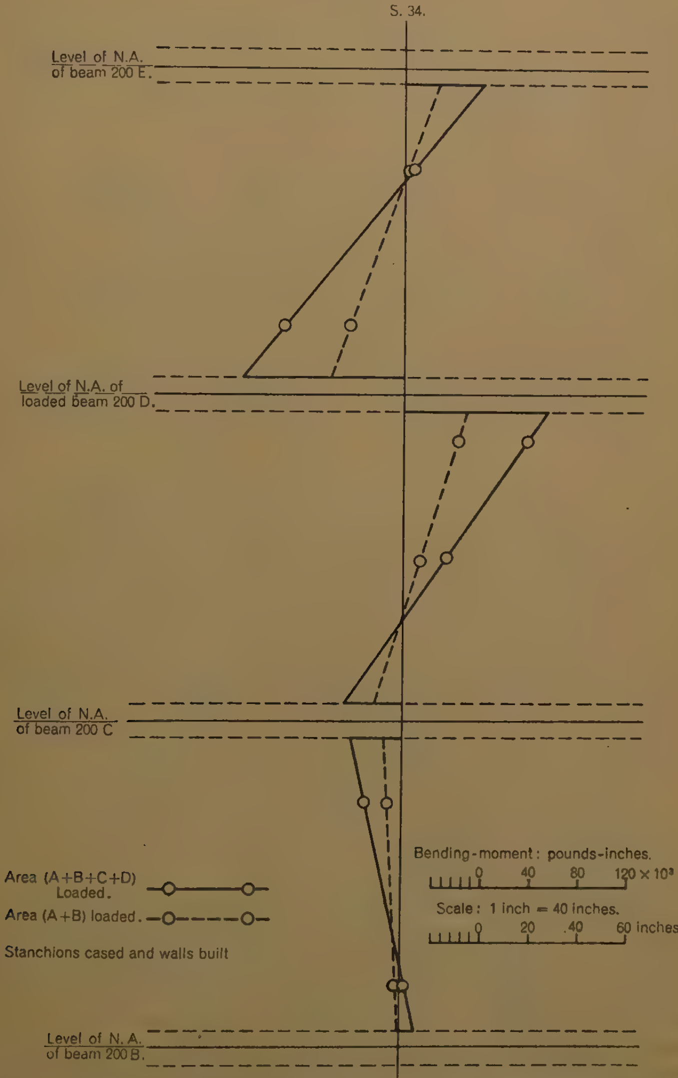
Accuracy of Observations.

A true assessment of the dependability of the observations is a matter of the first importance in work of this kind, where the actual behaviour of a structure is being determined. Great attention was

[1] Final Report of the Steel Structures Research Committee, published by H.M. Stationery Office, 1936. (Issued since the discussion on this Paper.—Sec. Inst. C.E.).

paid to this, and, although individual errors of from 150 to 300 lbs. per square inch were found in certain of the observations, there is

Fig. 12.



reliable evidence to show that the error in the bending stresses, from which most of the deductions were drawn, was much less, certainly not exceeding 100 lbs. per square inch.

General Behaviour of Frames.

The three buildings tested behaved normally under load in that appreciable restraining moments were developed at the ends of the beams, with corresponding bending moments in the stanchion-lengths. It was found that the method of stress-analysis developed could be made to give a reliable estimate of the stresses in a frame even when it was clothed.

The magnitudes of the restraining moments can be gauged from the fact that the "equivalent eccentricity" of a connection (defined as the distance from the centre-line of the stanchion at which the reaction from a similarly-loaded simply-supported beam would have to act to produce a moment equal to that observed), lay between 30·0 and 44·6 inches for the bare frame of the hotel building, between 16·0 and 34·3 inches for the office building, and between 5·2 and 11·0 inches for the flats building. These restraining moments, while reducing the maximum stresses in the beams by from 17 to 25 per cent. of those which would be found in similarly-loaded simply-supported beams, were at the same time responsible for large bending stresses in the stanchions, in some cases as much as 9 times those which would be estimated by the existing methods of design.

While sway of the bare unsymmetrical frames was detected, it was fortunately eliminated after the addition of floors and casing. The presence of sway makes it difficult to give any simple expression for the bending moments in the members of a frame adjacent to a loaded beam. In frames which are not to be clothed, therefore, great care must be exercised in formulating design-rules, particularly for stanchions, since the distribution of bending stresses will depend very largely on the proportions of quite distant members. Evidence, both from these tests and by calculation, shows that it should be safe, in all but the most extreme cases, to neglect sway when the frame has floors of hollow tile or similar construction, brick walls and even light stanchion-casings.

The opportunity was taken, after the hollow-tile floors had been laid in the buildings, of measuring the stresses induced in adjacent stanchions by the application of a central concentrated load to a beam not framing into those stanchions. These stresses, produced by the slab effect of the floor, were appreciable. The bending stress developed in an adjacent stanchion was found to be as much as 20 per cent. of the stress in the stanchion into which the loaded beam framed. Considerably more data will be needed before advantage can be taken of the fact in a design-method, but there is no doubt that, as far as the stresses in the stanchions are con-

cerned, each bay of a hollow-tile floor should be considered as a slab capable of bending a stanchion about both principal axes.

Among the many smaller matters illustrated by the tests was an indication of the considerable local stress which can be set up in a stanchion by unequal bearing on its base. The foot of one stanchion in the hotel building rested on a 5-inch-thick steel slab. Strain-readings taken 12 inches above the foot of the stanchion showed that the stress at one corner of a flange was 17 times as great as that at any other corner. Examination of the base showed that the holding-down bolts had not been screwed home as far as they might have been. A further test, made after these bolts had been tightened, showed that some redistribution of stress had taken place over the cross section but that the stanchion was still unevenly bedded, since at the one corner the stress was still more than 3 times that at any other. The load applied to this stanchion-length by the placing of the floors must have been sufficient to produce local yielding in the foot of the stanchion, as when a test was carried out after the floors were in position the distribution of stress at the section 12 inches above the base was quite normal.

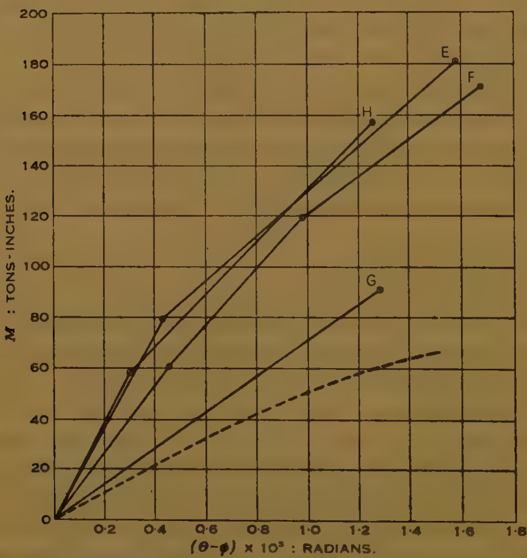
Behaviour of Bare Connections.

One of the main objects of the tests was the collection of data which would throw light on the variability of the behaviour of beam-to-stanchion connections made to the same design under normal conditions on the site.

If, in designing the beams, advantage is to be taken of the restraining moments transmitted by the connections, a very complete knowledge of the characteristics of the connections must be obtained. This has been done for the Committee mainly by means of laboratory tests, as will be explained later, where a number of specimens with connections made to the same design by different fabricators was tested, so that a measure of the variability of behaviour could be made. It would have been impossible, however, to depend on this information alone. One of the most probable causes of variability appeared to be differences in workmanship. The method of fabricating the laboratory specimens was by no means the same as that which would normally be used on the site. Apart from the more rigorous conditions there, it is usual for stanchions to be erected and for a beam to be brought between them, the connection at one end being made before that at the other. While the first connection to be made might be represented faithfully enough by a laboratory specimen, the conditions of fabrication at the other end would in all probability be so different that appreciable variation in behaviour

might result. The only way of investigating this was by tests on connections to the beams of actual building-frames. The most satisfactory method of comparison is to plot (as was done for the laboratory tests) the relation between the moment transmitted by each connection and the relative rotation $(\theta - \phi)$ of the members joined. These curves are shown in *Fig. 13*, for the connections at levels E, F, G and H in the single-bay portion of the hotel building, which were made to the same design (*Fig. 14*). It can be seen that, whilst there was little variation in the behaviour of the connections at levels H, E, and F, that at level G was appreciably different. The

Fig. 13.



curves for three connections to the internal stanchion of the hotel building (*Fig. 15*) are given in *Fig. 16* (p. 160). The curves for the two classes of connection (*Figs. 17* and *18*, p. 161) found in the office building are given in *Figs. 19* and *20* (pp. 162 and 163). In all these cases, for both the hotel and office buildings, the agreement between the curves, with the exception of that at level G, hotel building, is remarkably good. There is no indication that the effect of workmanship on the site is likely to cause greater variation than that found as a result of the laboratory tests, from which the lower-limit curve, shown dotted in *Figs. 13* and *16*, has been drawn.

The curves for the connections of the residential-flats frame showed a very much greater variation. The connections were made

Fig. 14.

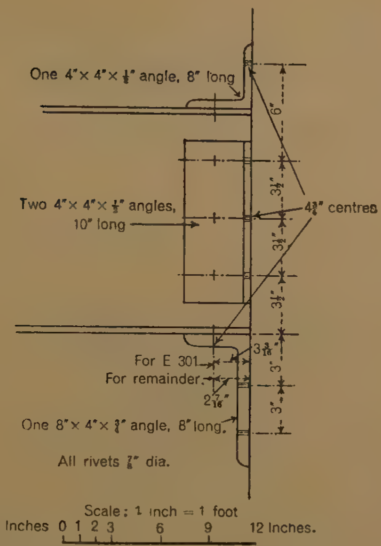
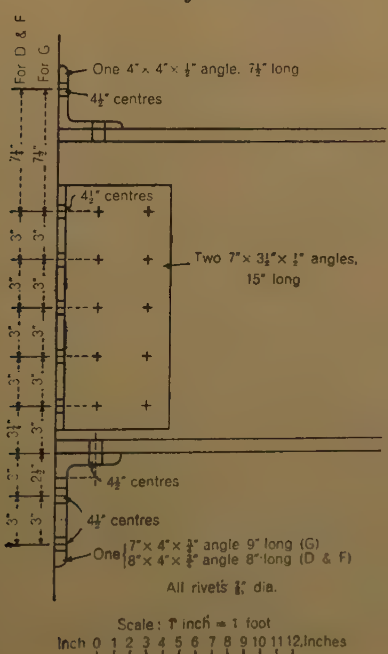
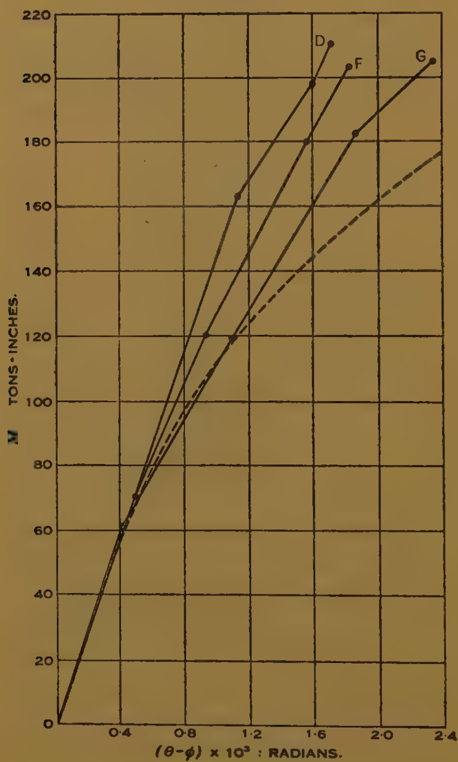

One 4"× 4"× ½" angle, 8" long
Two 4"× 4"× ½" angles,
10" long
4½" centres
6"
3½"
3½"
3½"
3"
3"
For E 301
For remainder
One 8"× 4"× ⅜" angle, 8" long
All rivets ⅞" dia.
Scale: 1 inch = 1 foot
Inches 0 1 2 3 6 9 12 Inches.

Fig. 15.


For D & F
For G
One 4"× 4"× ½" angle. 7½" long
4½" centres
7½"
7½"
4½" centres
Two 7"× 3½"× ½" angles,
15" long
3"
3½"
2½"
4½" centres
4½" centres
One 7"× 4"× ⅜" angle 9" long (G)
One 8"× 4"× ⅜" angle 8" long (D & F)
All rivets ⅞" dia.
Scale: 1 inch = 1 foot
Inch 0 1 2 3 4 5 6 7 8 9 10 11 12 Inches

up of 4-inch by 5-inch by $\frac{3}{8}$-inch top cleats with $\frac{3}{4}$-inch black bolts. The difference in behaviour was undoubtedly due to slip of the bolts, as was found in the more extensive laboratory tests, and showed the difficulty of evaluating a restraining moment to be used in

Fig. 16.

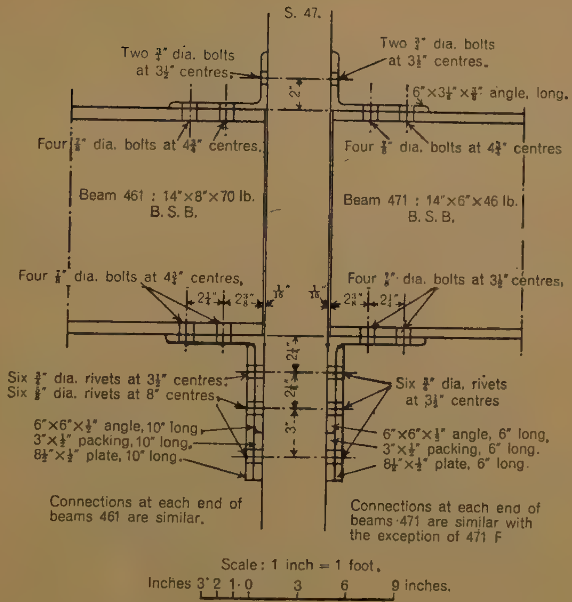


design when black bolts are used and not tightened to some definite torque.

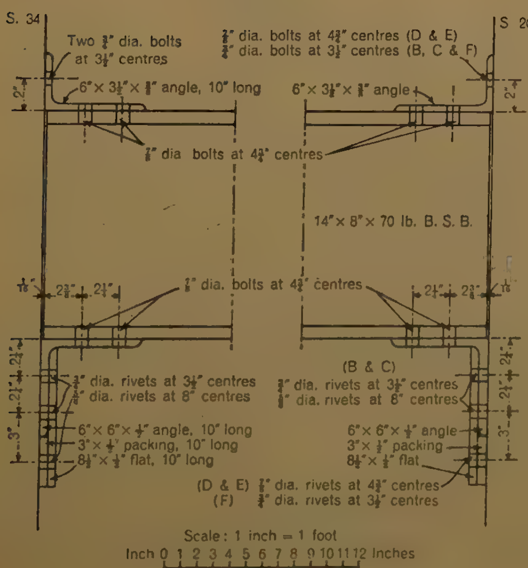
Cased Connections.

The curves given above have all referred to bare connections. In the great majority of important buildings the connections will be covered by the concrete of the floors or of the fire-resisting stanchion-casing. An attempt was made in the tests to find the effect of this concrete casing on the behaviour of the connections. It did not prove an easy matter, as the moment transmitted through the connection could only be deduced from the stresses produced in

Fig. 17.



Figs. 18.

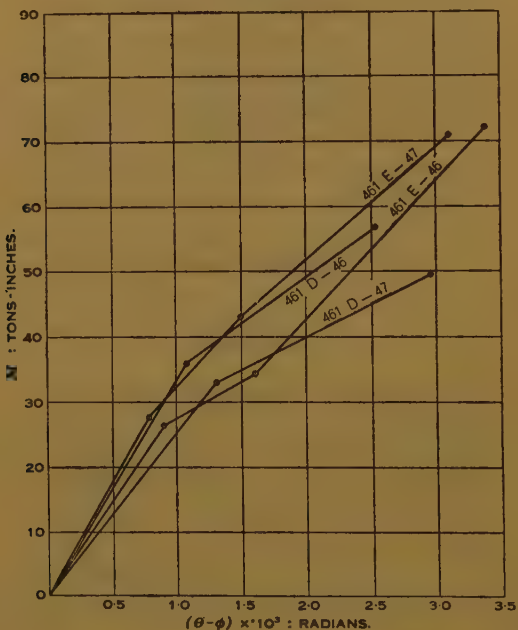


the stanchion, and it was found impossible to measure the relative rotation of the members. Characteristic curves could not, therefore, be drawn. A comparison of the stresses developed in the stanchions before and after the connections were cased gave some indication that the variability of the connections was decreased by the casing.

Partition of Moment Between Stanchion Lengths.

It is set out in Clause 44 (p. 132) of the existing method of design that the bending moment coming into a continuous pillar from a

Fig. 19.



loaded beam may be regarded as divided between the pillar-lengths above and below the level of the beam in direct proportion to the stiffnesses, that is (moment of inertia)/(length), of the upper and lower lengths.

The tests have shown that the actual partition of moment between the upper and lower stanchion-lengths is very different from that given by this rule. In all cases, except that of the internal stanchion of the hotel building, it was found that in the bare frame the lower stanchion-length received more of the bending moment coming

from the beam than this simple rule allows. Table V shows typical values of the true partition.

Fig. 20.

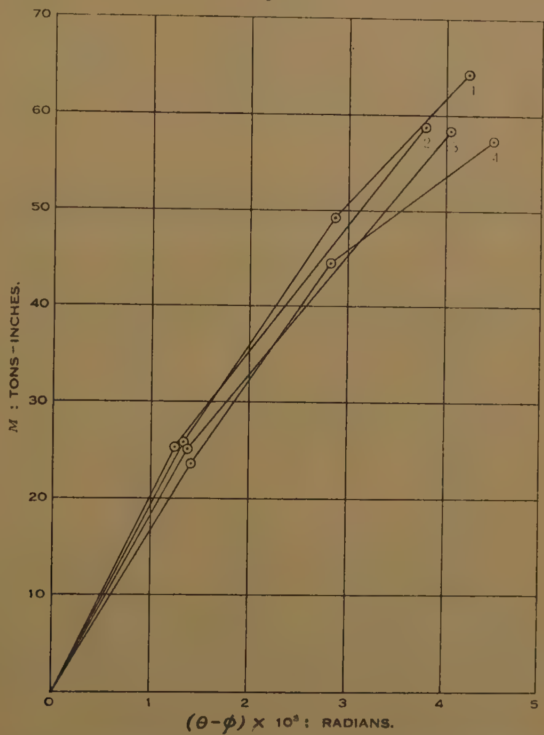


TABLE V.—A COMPARISON OF THE RATIOS OF STANCHION-STIFFNESSES AND BENDING MOMENTS IMMEDIATELY ABOVE AND BELOW THE NEUTRAL AXIS OF THE LOADED BEAM. (OFFICE BUILDING, SINGLE-BAY PORTION.)

Loaded beam.	Stanchion.	Ratio of stiffnesses of stanchion-lengths.	Ratio of bending moments.		
			With frame bare.	With floors laid.	With stanchions cased and floors laid.
200 D	S. 20	0·95	0·54	0·95	—
	S. 34	0·99	0·46	0·70	1·10
200 C	S. 20	0·78	—	0·47	—
	S. 34	0·94	—	0·73	0·96
200 E	S. 20	0·85	0·71	—	—
	S. 34	1·00	0·56	—	—

Fortunately, when the floors were laid and the stanchions were cased, the partition of moment approached much more nearly that of the ratio of the stiffnesses of the stanchion-lengths, as can be seen from Table V (cols. 5 and 6). In the internal stanchion of the hotel building the state of affairs was very different. It will be seen from Table VI that the upper stanchion-length received more than its share of moment and that the partition was not seriously modified when floors were laid.

TABLE VI.—A COMPARISON OF THE RATIOS OF THE STANCHION-STIFFNESSES AND BENDING MOMENTS IMMEDIATELY ABOVE AND BELOW THE NEUTRAL AXIS OF THE LOADED BEAM. (HOTEL BUILDING, INTERNAL STANCHION NO. 8A.)

Loaded beam.	Ratio of stiffnesses of stanchion-lengths.	Ratio of bending moments.		
		With frame bare.	With floors laid.	With stanchions cased and floors laid.
81 D	1·00	1·21	1·13	—
81 F	0·75	1·24	0·96	1·01
81 G	0·89	1·34	1·42	—
81 H	0·67	1·35	1·39	—

The "Code of Practice" rule (Clause 44, p. 132) was based on a consideration of the simplest possible frame in which the top and bottom ends of the stanchion-lengths were encastered and in which horizontal sway was prevented. In an actual frame there would be some rotation of the ends of the stanchion-lengths remote from the loaded beam, and in a bare frame there would also, in general, be some sway. Neither of these effects could, however, produce the change in the partition shown by the tests, and it must, therefore, arise from the axial thrust developed in a loaded beam by the behaviour of the end connections. If a beam connected to supports by a flange-cleat connection is considered, it will be seen that, as transverse load is applied to the beam, the bottom or tension flange tends to increase in length while the top or compression flange decreases. Owing to the fact that the distortion of the bottom brackets of the connections cannot take place as freely as that of the top cleats, thrust is developed in the beam if the supports are not perfectly free to move. This thrust could produce the ratios of moments observed in the stanchions of the single bay frames, where it was less than the ratio of the stiffnesses of the members.

In a multi-bay frame thrusts will be developed in both the loaded beam and in the beam on the other side of the internal stanchion to which they are connected. Where the sections of the beams are

not very different, the thrust in the loaded beam will be the greater and the effect on the moments in the internal stanchion will be similar to that in the stanchions of a single-bay frame. This was the case in the two-bay frame of the office building. Where, as in the multi-bay portion of the hotel building, the loaded beam is of a much smaller section than that on the other side of the internal stanchion, the thrust in it may be less than the thrust in the unloaded beam. This will produce a different form of partition of stanchion-moment, the ratio of moments above and below the level of the beam being greater than the corresponding ratio of stanchion-stiffnesses.

In the present state of knowledge it is impossible to predict with any accuracy the exact ratio of the stanchion-moments, but the important fact remains that, if a close estimate of the total moment coming on to the stanchion is made in design, then the use of the simple Code-of-Practice rule will, in most cases, result in an under-estimate of the maximum end bending stress in the stanchion, since the greatest total moment will, in general, arise when the heavier beam attached to the stanchion is loaded. This must be remembered when rules for design are drafted.

Effect of Floors.

It may be as well to point out that the placing of floors influences the stanchion-stresses in three ways.

(*a*) The presence of beam-casing or of hollow-tile floors increases the effective stiffness of the beam in the frame, and so, other influences being unaltered, tends to decrease the bending stresses in the stanchions.

(*b*) The concrete which is placed around the connections when the floors are laid increases, to some extent, the rigidities of the connections, with the result that the moments transmitted to the stanchions and the bending stresses induced in them tend to increase.

(*c*) The floors form slabs connecting all the bents of the framework, so that the effect of a load on one beam is felt not only in the stanchions to which the beam frames, as in the bare framework, but in adjacent stanchions also. Due to this slab-effect, therefore, the stanchion-stresses produced by the application of a concentrated load to a beam are likely to decrease.

It will be realized that it was not easy to study all these influences separately.

In the single-bay portion of the hotel building the addition of floors decreased appreciably the bending stresses induced in the stanchions when a central concentrated load was applied to a beam framing into them. In ten of the twelve stanchion-lengths tested the decrease in maximum bending stress varied from 37·5 to 21·1 per cent. Since in practice the applied load would be distributed over a considerable floor-area it was essential to evaluate the slab-effect. This was done by measuring the stresses in adjacent stanchions when a concentrated load was applied to a beam. From such measurements it was deduced that, in this particular structure, had a distributed load been applied to the whole floor-area the maximum stanchion bending stresses in the frame with floors laid would, with two exceptions, have varied between 10·8 per cent. less and 14·7 per cent. more than the corresponding stresses in the bare frame under the same load. In two stanchion lengths, however, increases of as much as 40 and 62·5 per cent. would have been found. Where the internal stanchion of the hotel building was concerned, the presence of floors decreased in every case the bending stresses induced when a concentrated load was applied to the beams framing into it.

A very different state of affairs was found in the office building. There, a considerable increase in stanchion bending stress was found when the floors were in position. As measured by the "equivalent eccentricity" of a connection, an increase of 53 per cent. was found in one stanchion of the single-bay portion when a concentrated load was applied to a beam, that is to say without allowance for slab-effect. When, as was fortunately possible in this building, a distributed load was applied to the floor so that the slab-effect was automatically included, the increase in stress was as much as 66·5 per cent. over that which would have been estimated in the bare frame carrying the same load.

When the floor was laid in the two-bay portion of the office building, casing was at the same time poured around the internal stanchion. In spite of this, and the fact that the floor was continuous past the internal stanchion, the increase of stanchion bending stress was no less than 61·3 per cent.

Although these effects are so different from those found in the case of the hotel building the reason is not difficult to find. It will be seen (*Figs. 14, 15, 17* and *18*) that in the hotel the bare beam-to-stanchion connections were much less flexible than those of the office building. The addition of concrete would therefore increase the rigidity of the former far less than that of the latter and would make it much more likely for the decrease in stress, due to the added stiffness of the beams, to prevail.

Effect of Stanchion-Casing and Walls.

Floors are not the only clothing likely to affect the stresses in the steel framework; the fire-resisting casing added to the stanchions and the external walls will also play their part.

Apart from any effect due to the reduction in sway, the presence of casings and walls will tend to stiffen the stanchions and so increase the moments developed in them. These increased moments may not, however, produce increased bending stresses in the steelwork, since the total section of the stanchion is increased by the addition of casing. For the investigations of these effects additional tests were carried out after walls and stanchion-casings had been built.

Although the stanchion-casing of the hotel building was very light in form, two tests made on the fully-clothed single-bay portion showed that its presence decreased the maximum bending stress in each stanchion-length below that observed when the floors alone were in place. The decrease in the five stanchion-lengths concerned varied from 10 to 35·5 per cent. In one test, strain-measurements were made on adjacent stanchions also, so that the slab-effect could be evaluated. It was found that the stresses in the bare frame would have been approximately 31 per cent. more than the corresponding stresses in the completely-clothed frame subjected to a distributed load. Only one test was carried out on the internal stanchion of the hotel building after the casing was in position. There, again, small but quite definite reductions in the stanchion-stresses were found. From this it appears that the addition of clothing to this particular frame relieved appreciably the stresses in the steel framework.

Once more, very different conditions were found in the office building. In one test on the single-bay portion the presence of walls and casing, which were more substantial than those of the hotel building, brought about an increase in the total bending moment in the steel core of 19 per cent. of the value it would have had in the frame with floors alone in position. In one stanchion-length the increase in the maximum bending stress was 47·2 per cent.

In the two-bay portion of the office building the increase in stress was more striking. In one length of the internal stanchion the maximum bending stress increased by 97 per cent. above the value it would have had in the frame without external walls and external stanchion-casing. In other lengths of the same stanchion the corresponding increases were 41 and 27 per cent.

These tests have shown that the effect of clothing cannot be ignored. In a frame such as the hotel building, with a rigid type of bare steelwork-connection, the effect of the clothing may, in general, be to reduce the stresses somewhat below the values which would be

estimated, by an accurate method of stress-analysis, in the bare frame. In a frame such as the office building, with more flexible connections, the stresses may be very appreciably increased. The bending stress found in a certain stanchion-length of the fully-clothed building was 2·89 times that which would have been set up in the bare frame. In other cases the increase in bending stress was as much as 60 per cent. It appears from this that, until more is known of the effect of cased connections, the assumption must be made in the design of stanchions that, where a frame is encased in concrete, all connections are perfectly rigid.

Detailed Investigation of the Behaviour of Connections.

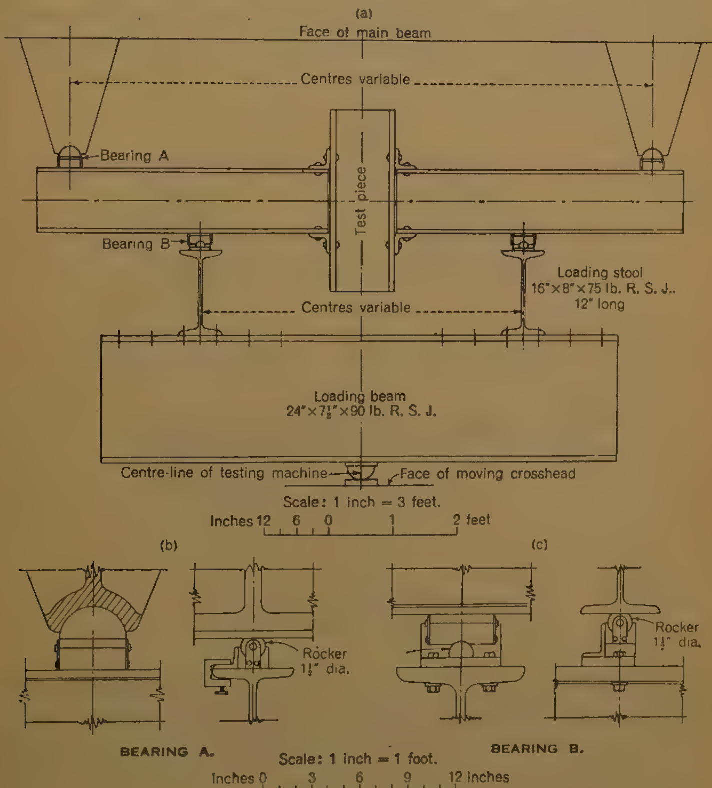
Laboratory Tests on Connections.

While the tests on existing buildings were being carried out, a detailed investigation of the behaviour of typical beam-to-stanchion connections was made in the Civil Engineering Laboratory of Birmingham University by Professor Batho. It is impossible to do full justice here to that work, and a short outline only will be given, sufficient to show the more important results.

The form of specimen used by Professor Batho consisted of two cantilever-beams attached to the opposite flanges of a short length of rolled-steel joist, representing the stanchion, by the connections to be tested. The specimen was placed in a testing machine, *Figs. 21*, and loaded as a beam. Different methods of loading were used so that the connection could be subjected to pure moment or moment combined with shear. Observations taken during the tests consisted mainly of measurements of the relative rotational or linear movements occurring between different parts of the specimen, by means of mirrors, extensometers and other devices, including a special form of extensometer devised for measuring the extensions of the bolts or rivets. These observations and the tests on members of the existing buildings showed that, when a beam having end connections of the simple bottom-bracket and top-cleat type (*Figs. 22*, p. 170) is loaded, a curve similar to OAB, *Fig. 23* (p. 171), represents the relation between M, the moment transmitted through a connection, and $\theta - \phi$, the relative rotation of the members joined by the connection. It may be well to point out that had the members been joined by a free pin the curve would have coincided with the axis OX, and had they been rigidly connected it would have coincided with the axis OY. When load is removed from the beam the curve traced out is BCDE. The moment has dropped to zero at D before all the load has been

removed from the beam. As the remainder of the load is removed a reversed moment comes into operation (D–E) until, when the beam is once more unloaded (E), there is left an end moment OM tending to cause sagging of the beam and a residual relative rotation ON. On reloading, the curve traced out will be EFB, the point B being reached under the same load as before. If further load is now added the first loading-curve OAB will be continued to G.

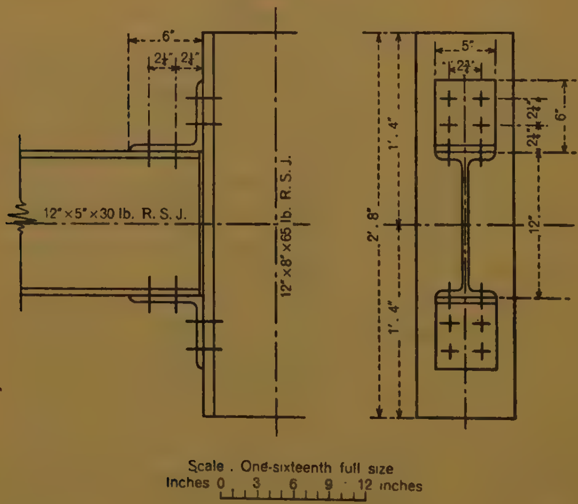
Figs. 21.



It will be seen from these curves that the connection is not elastic. This fact may have an important bearing on the behaviour of the frame as a whole, and some knowledge of the cause of the deformations suffered by the connection is essential to the designer. A complete analysis of the behaviour of a connection is difficult, and no attempt will be made here to give more than a descriptive treatment.

When a small moment is transmitted through the connection, due to the application of load to the beam, the legs of the bottom bracket and top cleat forming the connection will bend as if they were beams clamped firmly to the members at the rows of rivets nearest the roots of the angles. If the vertical leg of the bottom bracket were perfectly fitted to the face of the stanchion, it would be unable to bend. It is always found in practice, however, that the fitting is not good, and under small transmitted moments the bottom bracket bends as freely as the top cleat so that the end of the beam rotates about its centre of depth. In these circumstances the relation between rotation and transmitted moment is a straight

Figs. 22.

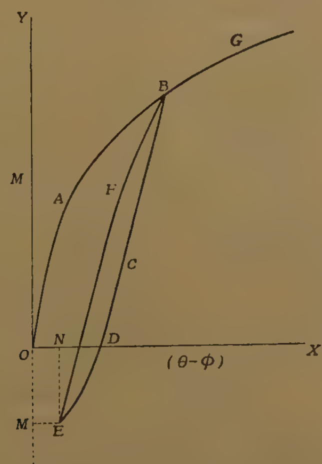


line. As the moment transmitted increases the pull in the rivets connecting the vertical leg of the cleat to the stanchion increases. These rivets clamp the cleat efficiently until the pull in them exceeds their initial tension, when they begin to extend elastically, increasing the flexibility of the connection. This continues until, as the moment increases further, the distortion of the top cleat is such that yield occurs in it, when the flexibility increases still more. This last is the inelastic range, the rate of change of moment decreasing markedly.

In actual fact the behaviour is much more complex than that described above. In no part of the range does there appear to be a true linear relation between moment and rotation; some inelastic

deformation occurs under the smallest moment. Other influences are the closing in of the vertical leg of the bottom bracket and the relative movement between the horizontal leg of the cleat and the beam. As the vertical leg of the bracket bends it comes sooner or later into contact with the face of the stanchion and is restrained; thus after a certain, but usually small, transmitted moment has been reached, the centre of rotation of the end of the beam moves down from the axis of the beam until eventually, when the whole of the vertical leg of the bracket is in close contact with the stanchion, it is at the level of the bottom flange of the beam. This change in the position of the centre of rotation tends to increase the rigidity of the connection. Relative movement between the horizontal leg

Fig. 23.



of the cleat and the top flange of the beam may occur. Sometimes, more particularly when bolts are used, this "slip" is sudden, the relative rotation increasing considerably for a very small increase in transmitted moment.

So far only the deformations in the cleats and rivets or bolts have been considered. The local deformations in the members to which they are attached may also be of considerable importance. The action on the stanchion of a bracket-and-cleat type of connection may be taken to be the same as that of two equal and opposite horizontal forces applied in the neighbourhood of the rivets through the vertical legs of the bracket and cleat. The strains and stresses in the stanchion due to those forces can be estimated approximately. It will be found, when the connection of the beam is to the flange of

a stanchion, that the strains, which always tend to increase the flexibility of the connection, are too small to have any appreciable influence on the stress-distribution in the member at sections remote from the connections. The local stresses can, however, be considerable. When the connections are heavy the stresses developed in the web of the stanchion may be so large as to call for the introduction of web-stiffeners. When a beam is connected to the web of the stanchion and is not balanced by a beam on the other side the local deformations of the web may be comparable to those of the connection itself. The overall flexibility may thus be considerably increased and the effect on the stresses in the frame marked.

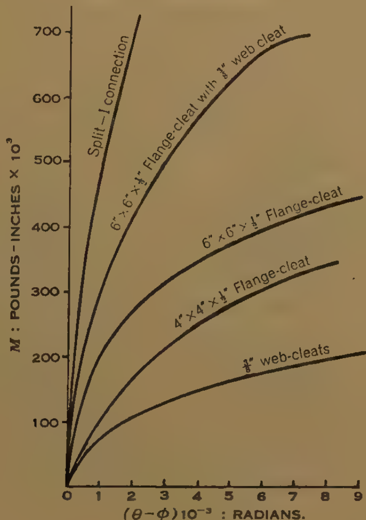
The part of the curve OAB (*Fig. 23*) produced on the first application of the load has now been considered in some detail. When load is removed the sequence of events is much the same, the deformations which occur first being similar to the "elastic" deformations produced on loading, so that the slope of the portion BC is not very different from that of OA. As more load is removed, the beam, in straightening further, deforms the bracket and cleat, and the slope of the curve decreases as from C to D. At D no moment is being transmitted through the connection, but, as the inelastic deformation produced on first loading has not been removed, there is still appreciable relative rotation of the members and there must therefore be some load still remaining on the beam. As this is removed the beam, in straightening, transmits a moment through the connection, of the opposite sense from that so far produced, and further deformation of the connection occurs until at the point E, when all load has been removed, the reverse moment is a maximum and there is some residual relative rotation. The magnitude of the reverse moment can be very much greater than that shown in *Fig. 23.* On reloading, the curve traced out is EFB, a closed loop being formed which is retraced almost exactly as further unloading and loading is carried out. If, after B has been reached, more load is added to the beam the first-loading curve OAB will be continued to some point G.

A number of first-loading curves for bracket-and-top-cleat (flange-cleat) connections are shown in *Fig. 24*, each curve being marked with the size of angle used for the cleats. In each case the connections were 5 inches long, the beam 12 inches deep and the rivets $\frac{3}{4}$ inch in diameter. Further tests were made on connections ranging in length from 5 inches to 8 inches. They showed that there was no marked increase in stiffness with length except in the case of $\frac{3}{8}$-inch-thick cleats. Another form of steelwork-connection consists of a pair of angles connecting the web of the beam to the stanchion. Such a web-cleat connection behaves in much the same way as the

flange-cleat type discussed above, but is more flexible. A typical curve for a web-cleat connection made up of 6-inch by $3\frac{1}{2}$-inch by $\frac{3}{8}$-inch angles 9 inches long, joined to a 12-inch-deep beam and to a stanchion by $\frac{3}{4}$-inch rivets, is shown in *Fig. 24*.

Though the web-cleat connection alone is very flexible the addition of angles connecting the web of a beam to a stanchion already joined by flange-cleats increases the rigidity of the connection considerably above that for the flange-cleats alone. This is shown in *Fig. 24* by

Fig. 24.



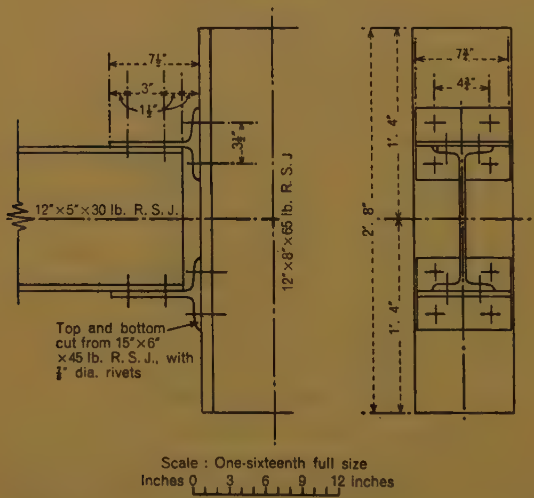
the curve for a connection made up of 6-inch by 6-inch by $\frac{1}{2}$-inch flange-cleats and $\frac{3}{8}$-inch web-cleats to a 12-inch-deep beam.

A type of connection sometimes used when heavy moments have to be transmitted is the split I-connection (*Figs. 25*, p. 174). Two large T-sections, made by cutting one flange from a length of I-section, join the top and bottom flanges of the beam to the stanchion. It will be seen that this type of connection is likely to be more rigid than that made up of flange- or web-cleats. Until the moment transmitted is considerable the only deformation which can take place is stretch of the rivets through the flange of the split I-section, and the small amount of flexure of this flange. A typical curve for a split I-connection made with T-sections cut from a 15-inch by 6-inch by 45-lb. R.S.J., $7\frac{3}{4}$ inches long and fitted with $\frac{7}{8}$-inch rivets, is shown in *Fig. 24.* It will be seen that the relation between moment and relative rotation is almost linear, and it continues so

until a moment of 9×10^5 lb.-inches is reached. Above this the rate of increase of moment decreases, due probably to slip between the web of the split I-section and the flange of the beam.

Steelwork-connections differ in one important respect from the other parts of the structure of which they form part. They are, under normal loading conditions, overstrained, that is to say the yield-point of the material has been passed. This is no source of danger when the material used is as ductile as structural steel. Numerous tests to destruction have shown that the relative rotation needed to cause fracture is many times greater than that which can occur under normal circumstances in a steel frame. An important

Figs. 25.



deduction from these tests is that the connections may be treated as units and the application to them of the rules limiting working-stresses is unnecessary.

The curves shown in *Fig. 24* are for bare connections. Laboratory tests on encased connections show that the flexibility is reduced to a very small value until a moment is transmitted which cracks the concrete ; under greater moments the curve is similar to that for the uncased connection.

The most important result of this work, as far as design is concerned, was the production of lower-limit curves for a number of standard uncased connections. A comparison of the curves obtained by testing a number of connections of the same design made by different fabricators showed that there were variations due to

workmanship and other causes. Further investigations were carried out on the effects of the different factors producing variation, such as differences in the initial tension in the rivets or bolts, creep between the beam and the cleat, and abnormalities in the form of the cleat. As a result of these it was found possible to lay down with confidence a single curve similar in form to those shown in *Fig. 24* as the limit below which no connection of that particular design would be found ; it showed the least rigidity which could be depended upon. Such curves were drawn for a number of standard connections and formed the basis on which the method of design for beams was based.

When the characteristic curves (the relations between relative rotation and transmitted moment) for the connections at the ends of a beam are known, the restraining moments there under any loading on the beam can be found most conveniently by a graphical method due to Professor Batho. The simplest case is that of a beam uniformly loaded and having similar restraints at the ends.

Let it be assumed that a beam of uniform cross section, relevant moment of inertia I and length l carries a uniformly-distributed load of intensity w per unit length, and is connected to rigid stanchions or abutments by identical connections, having a flexibility defined by the curve OPQ (*Fig. 26*, p. 176).

If ϕ is the change of slope of the end of the beam and M is the moment at the end of the beam, then, from the well-known slope-deflection equation for a beam of uniform cross section, the relation between the end moment and the end rotation is

$$M = \frac{2EI}{l}[\phi] - \frac{wl^2}{12}.$$

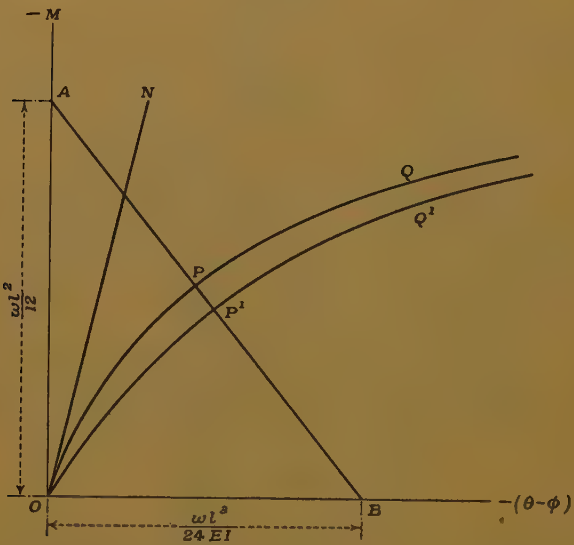
This is a straight line, and since θ, the change of slope of the stanchion, is zero, can be drawn as AB in *Fig. 26*. The curve OPQ, giving the relation between transmitted moment and angular rotation for the connection, is already plotted there, so that P, the intersection of AB and OPQ, gives the moment present at the end of the beam. The bending moment and the bending stress at any point in the beam can then be determined.

It is much more usual for a beam to be connected at its end to a stanchion which is flexible and will therefore change its slope when a moment is applied to it. Suppose the beam considered in the last example is connected at its ends to identical stanchions, the flexibility of which is known, so that in *Fig. 26* it is possible to plot the line ON showing the change of slope θ of a stanchion at the level where the beam is attached due to any moment applied to it at that point. Then if, as before, the curve OPQ defines the flexibility of

the beam-to-stanchion connection, the change of slope suffered by the end of the beam, when any moment M is transmitted from it through the connection to the stanchion, is given by the sum of the abscissæ of ON and OPQ corresponding to M. The curve representing this sum is OP′Q′, and, as in the first example, the moment at the end of the beam when a uniformly-distributed load of intensity w is applied is given by the intersection of OP′Q′ with the beam-line AB.

The constructions for unsymmetrical loading and different re-

Fig. 26.



straints at the ends are more complicated. They are given in full in the Steel Structures Research Committee's Final Report.

Use of Bolts.

Other investigations on joints were carried out by Professor Batho,[1] one of which it is convenient to mention here, namely, the behaviour of bolts and bolted connections. It would be of considerable advantage in the erection of steel-framed buildings if black bolts with the usual clearances could be used in place of rivets. The danger of slip taking place has made the use of black bolts undesirable except in those positions, such as the vertical leg of a top

[1] "Experimental and Theoretical Investigations on Riveted and Bolted Joints with the Application of the Theory to Welded Joints," First Report of the Steel Structures Research Committee (1931), p. 113.

cleat in a beam-to-stanchion connection, where they could be considered as subjected to tension only. Where the use of rivets has been impossible in erection it has been the custom to specify turned and fitted bolts, which are expensive.

The main difficulty preventing the use of black bolts was the impracticability of tightening them up sufficiently to ensure that slip did not occur at reasonable working-loads. It was found by tests, however, that bolts made from a high-tensile steel in accordance with the B.S.S. for high-tensile structural steel N548 (1934), could be tightened up sufficiently to give a reasonable margin of safety against slip and that there was no danger of the tension in a bolt being lost during tightening to torques considerably beyond the yield-point so long as the material of the bolt was ductile.

Professor Batho has found it possible to suggest the following clauses to govern the use of black bolts :—

Clause (1). The maximum torque which can be applied to a black bolt may be taken as

$$t\left(\frac{D}{D_0}\right)^3 \text{ lbs.-inches,}$$

where D is the nominal diameter of the bolt, D_0 is $\frac{3}{4}$ inch and t is the maximum safe torque on a bolt of the same material $\frac{3}{4}$ inch in diameter. (It is 2,000 lbs.-inches for ordinary mild-steel (from 26 to 28 tons per square inch) black bolts and 3,000 lbs.-inches for Chromador-steel black bolts.)

Clause (2). The safe resistance, S, to slip of a bolt may be taken as given by

$$\frac{S}{A} = 3{\cdot}6bc \text{ tons per square inch,}$$

where A denotes the nominal cross-sectional area of the bolt;

b „ ratio of torque applied to the bolt to the maximum safe torque for an ordinary black bolt of the same diameter, as given by clause (1).

c is a constant, with a value of 1·0 for joints in double shear, 0·75 for joints in single shear and 0·5 for joints in either single or double shear having oiled or painted plates.

From these clauses it will be seen that the torque applied to the bolt must reach a given value. The use of a special spanner which enables the torque to be controlled is therefore essential. Two types of spanner which meet this requirement have been described in the Committee's Second Report.[1]

[1] Published by H.M. Stationery Office, 1934.

12

The Problem of Design.

The more important points brought out by the experimental investigations can be summarized as follows :—

(*a*) Standard types of riveted and bolted beam-to-stanchion connections are capable of transmitting large bending moments, with the result that appreciable restraining moments are developed at the ends of a loaded beam in a steel-framed building. The connections themselves may be treated as units, and the relatively large stresses developed in them by the moments are not a source of danger.

(*b*) Although there was some variation due to workmanship in the behaviour of connections made to the same design, it was possible to set out a lower-limit curve for each type of connection, enabling a safe estimate of the restraining moment to be made.

(*c*) The moment transmitted through a connection from a loaded beam develops bending moments in the stanchion which are many times greater than those taken into account to-day.

(*d*) The bending moments developed in a stanchion due to load applied to a beam are appreciable, not only in the stanchion-lengths to which the beam is attached, but also in those more remote.

(*e*) The ratio of maximum bending moments in the stanchion-lengths above and below a loaded beam can be very different from that of the stiffnesses of the stanchion-lengths.

(*f*) The addition of floors, walls and stanchion-casing to a frame does not bring about any fundamental change in the behaviour of the frame.

(*g*) Concrete casing can increase the rigidity of connections appreciably in the working-range.

(*h*) The presence of clothing decreases the maximum stresses in the steel beams of a frame but it may increase the bending stresses in the steel cores of the stanchions.

(*i*) The methods of stress-analysis developed could be depended upon to give a true picture of the distribution of stress in the steelwork of a frame even when it was clothed.

The investigations thus show that the stress-calculations made in

design to-day give a very faulty representation of the distribution of stress in a frame. They emphasize the shortcomings in stanchion-design which have been mentioned on p. 128, and draw attention to the extravagance of the present method of design of beams, in which restraining moments are neglected. What is still more important, however, they disprove the usual assumption that the worst conditions are provided for each member when every member carries its full load.

It is not easy to calculate the stresses in such a complicated structure as a building-frame. Simplified assumptions must always be made by the designer, but if those assumptions are so sweeping that the true behaviour of the frame is disguised, then economy of material and evolution of the method of construction are impossible. The Steel Structures Research Committee has endeavoured to produce a method of design which, while simple enough to be applied in ordinary practice, is based on an accurate estimate of conditions in the structure.

The tests on existing buildings show that the stresses in a member of a steel building-frame are influenced not only by the size of that member and the load which it carries, but also by the proportions and conditions of the adjacent members and of the connections between them. It is not difficult to see that the restraining moment developed at the end of a loaded beam joined to a stanchion by a certain connection depends not only on the characteristics of the connection but on the stiffness (the relevant moment of inertia divided by the length) of the stanchion. If the stanchion is so stiff that it can be assumed to be rigid, then the restraining moment developed will have its maximum value; as the stiffness of the stanchion decreases so does the restraining moment. The maximum stress in the beam is influenced by the magnitude of the restraining moment, so that the suitability of a beam to carry a given load depends on the connections at its ends and on the stanchion to which it is attached. In the same way the bending moment applied to the stanchion by a loaded beam is influenced by the proportions of the beam.

As a designer cannot afford to make a tentative design of the whole structure and then to modify each member as the effect of the remainder of the frame is appreciated, some means had to be found which would enable one member to be designed economically without reference to the actual properties of the rest of the structure. It was found that, whilst the stresses in each member are affected to some extent by the conditions of the rest of the frame, the maximum stress in a loaded beam fitted with a certain semi-rigid beam-to-stanchion connection is not particularly sensitive to changes in the stiffnesses of the members into which it frames. A safe design

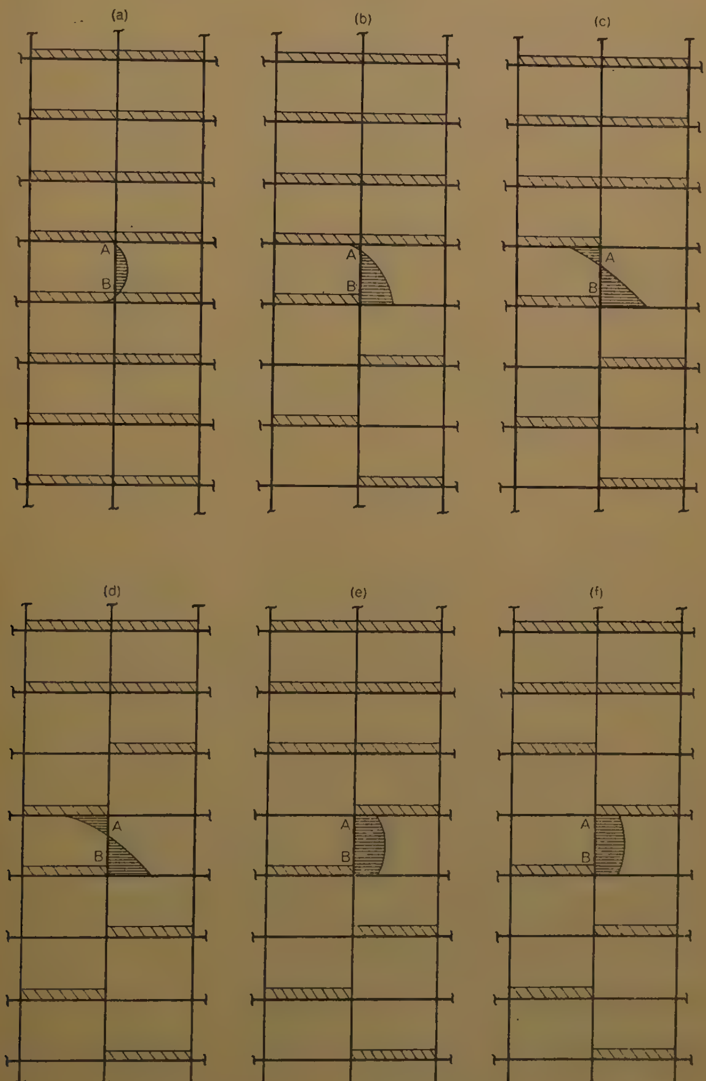
without serious loss of economy would, therefore, result if a lower limit were assumed for the stiffnesses of these members when the restraining moment on the beam was estimated. From the evidence supplied by a large number of frames designed under existing methods it was seen that the sum of the stiffnesses of the stanchion-lengths into which a beam framed would not, except in special cases, be less than two-thirds of the stiffness of the beam. Assuming the stiffness of the stanchion-lengths to be two-thirds that of the beam, it was possible to find, by the graphical method described on p. 168, the restraining moments at the ends of any loaded beam fitted with any type of standard connection, and therefore, taking account of the relief given by these moments, to design a beam which would be safe and more economical than those used to-day, whatever the actual sizes of the adjacent members in the structure. With the beams designed in this way, the design of the stanchions could be approached with some confidence.

The difficulty in designing a stanchion-length is to decide what conditions give rise to the greatest stress in the member. In *Figs. 27* are set out diagrams showing the bending stresses in a symmetrically-placed stanchion-length, AB, deduced from the results of the tests on existing buildings, due to various arrangements of superimposed load on the beams. In the existing method of design it is usual to choose a stanchion-section from a consideration of the conditions existing when all floors are loaded, *Figs. 27* (*a*), making the usual reduction in live load allowed in Clause 35 (*e*), p. 129. It is quite clear, however, that this load will not give rise to the greatest stress in the stanchion, since, by removing load from the floors below on alternate sides of the stanchion, as shown in *Figs. 27* (*b*), considerable end bending moments are developed in AB. This removal does not reduce the magnitude of the end load on AB but it does appreciably increase the maximum bending stress in the stanchion-length.

The conditions shown in *Figs. 27* (*b*) do not necessarily develop the absolute maximum stanchion-stress. If load is removed from part of the floor immediately above the stanchion-length as shown in *Figs. 27* (*c*) the end bending stresses are increased further and the decrease in axial end load arising from the removal of the load from the floor above may not compensate for this increase, leaving the maximum total stress greater than in *Figs. 27* (*b*). A further increase in end bending stress arises if other floors are unloaded as in *Figs. 27* (*d*), but in most structures the reduction in axial end load which follows outweighs this further increase. In these last three arrangements of load, shown in *Figs. 27* (*b*), (*c*), and (*d*), the bending moments applied by the beams to the ends of the stanchion-length bend it in

"double curvature." Another condition has to be considered in which the stanchion-length bends in "single curvature." An

Figs. 27.



arrangement of load which brings this about is shown in *Figs. 27* (*e*) together with the resulting form of the bending-stress diagram.

As before, the removal of further load (*Figs. 27* (*f*)) increases the bending stress but decreases the axial stress.

It is impossible to say from inspection which of the arrangements of load shown in *Figs. 27* (*b*)-(*f*) will give the absolute maximum stress in the stanchion-length, but it is certain that one or other of them will give a greater value than the arrangement of load hitherto used (*Figs. 27* (*a*)). It is clear that, for a complete treatment, the behaviour of a stanchion-length under axial end-load and end-moments, producing both " single " and " double curvature," must be studied. The arrangement of load which must be used in design depends on the relative values of the end-moments for the various cases which, in turn, depend on the layout and dimensions of the frame concerned. This will be dealt with later.

The design of a stanchion-length, therefore, resolves itself into two steps, (1) the determination of the end-reactions applied to the member by that arrangement of load on the structure which produces the worst conditions in the member, and (2) the estimate of the maximum stress developed in the member by those reactions, which enables the suitability of the member to be judged.

Evolution of Method of Design.

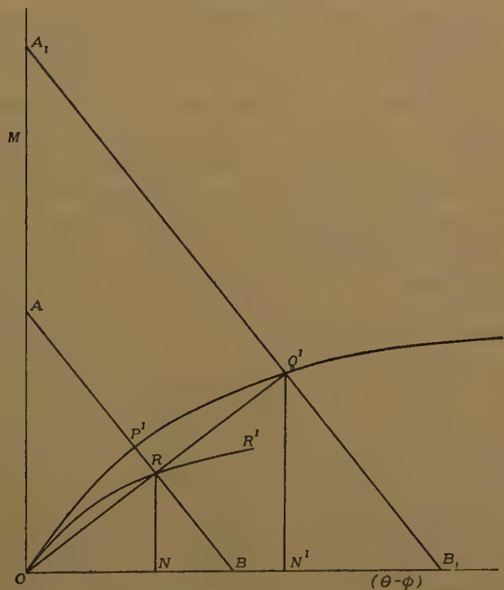
Load-Factor.

Before the details of design for either beams or stanchions could be settled, a method of obtaining a definite margin of safety in the structure had to be considered. It will be seen from the discussion of the behaviour of the beam-to-stanchion connections that if the load on a beam in a steel building-frame is doubled, the maximum flexural stress in the loaded beam occurring at some section remote from the ends is more than doubled. The same conditions exist in the stanchions as are recognized in the existing method of design (p. 132). To obtain an approximately uniform margin of safety it was decided, therefore, to adopt the basis of load-factor commonly used in aeronautical engineering, and to design each member so that if the design load was increased in a given ratio (the load-factor) failure would be imminent. The simplest way of ensuring this would be to design the structure so that when it carried a load equal to the design load multiplied by the load-factor, the failing stress of the material would be reached. For some reason this was considered, by those members of the Committee who represented the steel industry, to be too radical a departure, and the same result had to be obtained by subterfuge. The load-factor and failing stress of the material were taken to be 2 and 18 tons per square inch respectively.

Beams.

In the present method of design it is usual to assume a beam to be simply supported, to calculate the maximum bending moment produced in it by the application of the full dead and superimposed load, and to design the member for a given working-stress under this moment. It was considered desirable, as stated in the paragraph above, to retain this general method; that is, to design for a given working-stress when considering the maximum moment in the beam produced by the design loads. As a constant load-factor had to be

Fig. 28.



maintained it was impossible to use, in calculating the maximum moment, the real values of the restraining moments given by the lower-limit curves. The following method, due to Mr. E. W. Butler, of H.M. Office of Works, was used to obtain, from the lower-limit curves, other curves which would give the required factor of safety.

As explained on p. 168, a curve OP′Q′ (*Fig. 26*) can be drawn, giving the relation between the moment and rotation at the end of a symmetrically-loaded beam, from which, by the use of the beam-line AB, the restraining moment under the full design load can be found. If the load on the beam were doubled the restraining moment Q′N′ (*Fig. 28*) would be given by the new beam-line A_1B_1 parallel

to AB and such that $OA_1 = 2OA$. OQ′ is drawn, cutting AB in R. Then, if the end restraining moment allowed for in the design of the beam is RN, and if a nominal permissible flexural stress of 9 tons per square inch is worked to, the stress in the beam when the load is doubled will be 18 tons per square inch. A curve ORR′ can be drawn by this construction, to be used as the standard in design to replace the lower-limit curve.

The lower-limit curves, and those of the type ORR′, were deduced from the behaviour of connections on beams of 12 inches depth. Since it was found that the moment required to produce a given deformation in the connection was approximately proportional to the depth of the beam, and that the angle of rotation for a given deformation was very nearly inversely proportional to the depth of the beam, it was possible to plot from each curve another curve showing the behaviour of the same connection when attached to a beam of any depth. This was most easily done by plotting M/D as ordinates against $D\ (\theta - \phi)$ as abscissæ, where D denotes the depth of the beam; this gives a curve showing the relation between the nominal pull P, equal to M/D, on the top cleat and the relative linear movement of the ends of the upper and lower faces of the beam. The restraining moment provided by a connection attached to a beam of depth D can be found, using such a curve, by the beam-line method (p. 168), the intercepts on the axes being OA, equal to M_F/D, and OB, equal to $\dfrac{M_F Dl}{2EI}$, where M_F denotes the " fixed end moment " (that is, the moment present at the end of a similarly-loaded beam with completely fixed ends), and equals $\frac{1}{12}wl^2$ when the load is uniformly distributed and of intensity w. As the restraining moment is being determined so that the beam can be designed, it is undesirable to have I, the moment of inertia of the beam, appearing in the expression for OB. Trial designs can be avoided at a slight sacrifice of economy if a simplified method, devised by Professor Batho, is used, and a lower limit is taken for OB independent of the actual section of the beam.

If f_F is the maximum flexural stress in the beam-section produced by the moment M_F, then $M_F = \dfrac{2f_F I}{D}$

and
$$OB = \frac{M_F Dl}{2EI} = \frac{f_F l}{E}.$$

When a single concentrated load acts on the span, f_F has its lowest value, equal to $f_s/2$, where f_s denotes the maximum stress which would be found in the beam if it were simply supported. This

stress f_s cannot be less than 9 tons per square inch when the yield-stress of the material and the load-factor are taken to be 18 tons per square inch and 2 respectively, so that if, in drawing the beam-line, OB is made equal to $\frac{f_s l}{2E}$, f_s having the value of 9 tons per square inch, then a safe restraining moment is found, depending only upon the length of the beam, the depth of the section and the load carried, and expressed in the form of M_F. By drawing beam-lines in this way it is a simple matter to draw up tables or to plot families of design-curves for each type of connection showing the relation between $M_F/2D$, l and P (equal to M/D). Such design-curves for standard connections of classes "B" and "C", formed of 6-inch by 4-inch by $\frac{1}{2}$-inch, and 6-inch by 4-inch by $\frac{3}{4}$-inch flange-cleats, respectively, are given in *Figs. 29* (p. 186).

The argument given above was based on the consideration of a beam carrying a symmetrical load and with equal restraints at its ends. The results can, however, be applied when this symmetry does not exist. If the load is unsymmetrical but the restraints are equal, safety will be ensured if M_F is taken as the mean of the two fixed end moments, so that the design curve (*Figs. 29*) gives the mean of the end moments, which is taken to be the restraining moment at each end of the beam. When the restraints are unequal and the load is unsymmetrical an error on the unsafe side can occur, but even in an extreme case it is so small as to be negligible.

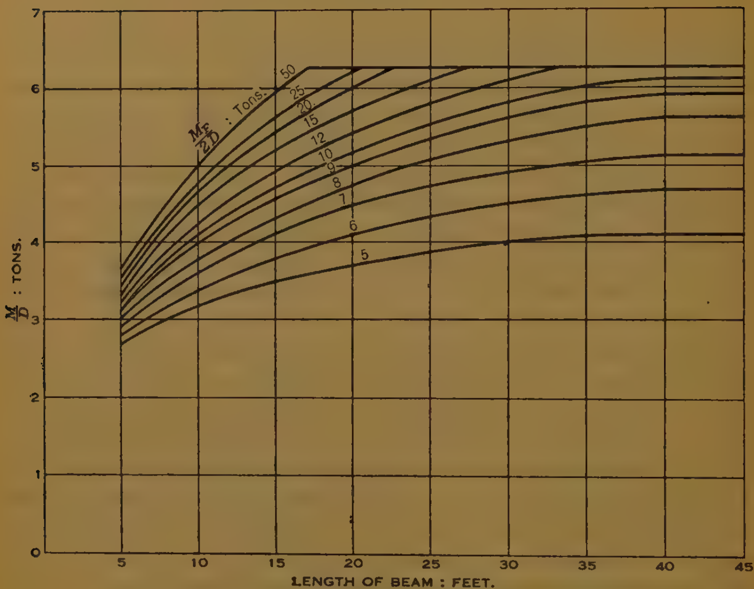
The lower-limit and standard curves used above give the restraint provided by a bare beam-to-stanchion connection. When the building is clothed so that the connections are encased in concrete it is possible that the restraint, under the design load, will be considerably greater than that shown by the curve. The lower limit for the cased connection depends upon the transmitted moment which produces cracks in the casing, and if there is no reinforcement this moment may be comparatively low. It was decided, therefore, that no allowance should be made for the extra restraint which might be provided by the casing.

It was possible, therefore, to draft the main clauses dealing with beams to be inserted in the rules for design. They are :—

> *Clause (2)*. Where a beam of uniform web depth throughout its length is attached at one end to the flange of a stanchion, allowance may be made for the restraining moment present at that end of the beam in the following manner :—
>
> The value of $\frac{M_F}{2D}$ shall be calculated, D being the depth

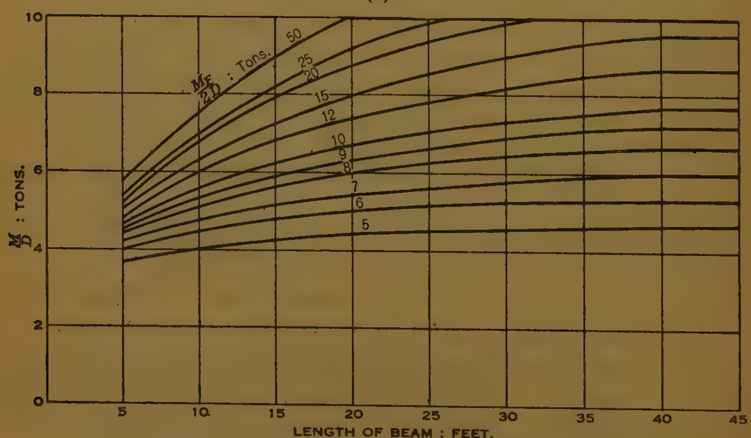
Figs. 29.

(*a*)



DESIGN OF BEAMS WITH CLASS "B" CONNECTIONS.

(*b*)



DESIGN OF BEAMS WITH CLASS "C" CONNECTIONS.

of the beam and M_F the mean of the fixed end moments (i.e. the moments present at the ends of a similarly loaded beam with completely fixed ends) and the appropriate table or curve (for example *Figs. 29*) used for the determination of the restraining moment M at the end of the beam.

Where in extreme cases, such as the upper storeys of certain frames, $\frac{K_B}{K_U + K_L}$ (K_B is the stiffness (I/L) of the beam and K_U and K_L are the stiffnesses of the stanchion lengths above and below the beam) is found to exceed 1·5, no allowance shall be made for the restraining moment unless the procedure of Clause 3 (1) is followed. In calculating the stiffness of a member the gross moment of inertia shall be used, no deduction being made for rivet-holes. For a plated beam the stiffness shall be taken as the maximum gross moment of inertia of the beam divided by its length.

Clause (*3*). Where a beam is connected to the web of a member, either stanchion or beam, no allowance may be made for the restraining moment at that end except in the following cases:—

(1) Where a beam frames into one side of the web of a stanchion, there being no beam on the other side, and special provision is made to stiffen the web of the stanchion, allowance may be made for the restraining moment provided that the value of $\frac{K_B}{K_U + K_L}$ does not exceed 1·5. If $\frac{K_B}{K_U + K_L}$ exceeds 1·5 the value of $q = \dfrac{1}{1 + \dfrac{2K_B}{3(K_U + K_L)}}$ shall be calculated, the restraining moment M being determined as in Clause 2, $\frac{qM_F}{D}$ being substituted for $\frac{M_F}{2D}$ in the appropriate table or curve (for example *Fig. 29*).

(2) Where the connection is balanced by a connection of the same class to a beam on the opposite side of the member, the rivets or bolts serving both connections, allowance may be made for the restraining moment

as set out in Clause (2), the fixed end moment being taken as that portion which is balanced by the fixed end moment due to dead load only in the beam on the opposite side of the member.

Critical Loading Conditions for Stanchion.

Reference has already been made (p. 178) to the arrangements of live or superimposed load which are likely to produce the most rigorous conditions of stress in a stanchion-length. The worst possible moment that can occur in any stanchion-length is made up of two parts, one due to the dead load on all the beams and the other due to the most unfavourable combination of live loads, *Figs. 27.* The first step in the production of a method of design is, therefore, the collection of data which will enable these moments to be estimated. The magnitudes of the moments are affected by the proportions of the members making up the frame and by the characteristics of the connections joining the members. The tests on existing buildings, while indicating that the methods of stress-analysis derived earlier in the investigation were reliable, showed that the casing of a connection could increase its rigidity very considerably. As the end moments in a stanchion-length increase as the rigidity of the connections between the members increases, it was decided that, for the normal type of clothed steel frame, it would be necessary, in estimating stanchion-moments, to assume that the joints in the frame were perfectly rigid. Making this assumption, which, while giving the maximum possible moments, had the additional advantage of removing one of the variables from the calculations, it was possible to draw up tables giving the desired information in a fairly compact form. Any exact determination of these worst moments can only be made, however, if the proportions of all the members in the frame are known. As the whole structure is not yet designed these proportions cannot be known, so that the data to be provided must be such that an upper-limit value of the moment can be estimated from the meagre knowledge of the frame already possessed by the designer. All that the designer has determined at this point is the sizes of the beams. It has been shown that the moments developed in the stanchions depend on the relation between beam and stanchion stiffness, so that, if an economical upper-limit value for the moment is to be found, the designer must, as he does in the existing method of design, first choose a stanchion-section and then, by the methods to be described later (p. 191), find whether his choice is satisfactory. The sections of the beams at one floor-level and of the stanchion-lengths above and below that level being known, it was found possible to compile

tables giving the maximum value of the bending moment which could be applied to the stanchion at that level, no matter what the arrangement of loaded beams or the proportions and layout of more distant members in the structure might be.

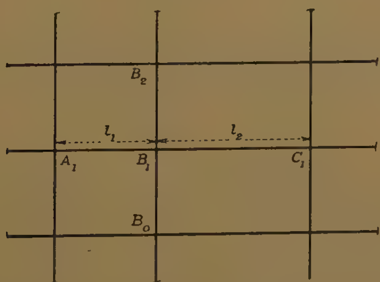
The calculations which had to be made before the tables of worst moments could be compiled were arduous and somewhat complicated. It is impossible in the space available here to give any details other than specimen entries. These are shown in Table VII and Table VIII (p. 190).

TABLE VII.—TOTAL BENDING MOMENT DUE TO DEAD LOAD.

$\frac{K_{BR}+K_{BL}}{K_U+K_L}$.	Bottom length; $\left(\frac{\text{total moment}}{M^D_{F_1}-M^D_{F_2}}\right)$.	Intermediate length; $\left(\frac{\text{total moment}}{M^D_{F_1}-M^D_{F_2}}\right)$.	Topmost length; $\left(\frac{\text{total moment}}{M^D_{F_1}-M^D_{F_2}}\right)$.
0·0	0·862	1·000	1·000
0·2	0·783	0·950	0·918
0·6	0·671	0·842	0·794
1·0	0·589	0·754	0·704
2·0	0·452	0·601	0·555
4·0	0·310	0·429	0·396
8·0	0·191	0·273	0·256

Each of the Tables is arranged in three main columns, referring to bottom, intermediate and topmost stanchion-lengths. The moment given by Table VII is the total moment coming on to the

Fig. 30.



stanchion at B_1 (*Fig. 30*) when all the beams carry dead load only. $\frac{K_{BR}+K_{BL}}{K_U+K_L}$ is the ratio of the sum of the stiffnesses of the beams A_1B_1 and B_1C_1 to the sum of the stiffnesses of the stanchion-lengths B_2B_1 and B_1B_0; $M^D_{F_1}$ and $M^D_{F_2}$ are the "fixed-end moments" at the

TABLE VIII.—TOTAL BENDING MOMENT DUE TO LIVE LOAD.

$\frac{K_{BR}+K_{BL}}{K_U+K_L}$	Bottom length.	Intermediate length.		Topmost length.	
	Double curvature.	Double curvature.	Single curvature.	Double curvature.	Single curvature.
0·0	$1{\cdot}149M\frac{L}{F_1}+0{\cdot}287M\frac{L}{F_2}$	$1{\cdot}359M\frac{L}{F_1}+0{\cdot}340M\frac{L}{F_2}$	$1{\cdot}000M\frac{L}{F_1}-0{\cdot}000M\frac{L}{F_2}$	$1{\cdot}000M\frac{L}{F_1}+0{\cdot}000M\frac{L}{F_2}$	$1{\cdot}000M\frac{L}{F_1}-0{\cdot}000M\frac{L}{F_2}$
0·2	$1{\cdot}017M\frac{L}{F_1}+0{\cdot}231M\frac{L}{F_2}$	$1{\cdot}230M\frac{L}{F_1}+0{\cdot}280M\frac{L}{F_2}$	$0{\cdot}887M\frac{L}{F_1}-0{\cdot}064M\frac{L}{F_2}$	$0{\cdot}925M\frac{L}{F_1}+0{\cdot}007M\frac{L}{F_2}$	$0{\cdot}891M\frac{L}{F_1}-0{\cdot}027M\frac{L}{F_2}$
0·6	$0{\cdot}834M\frac{L}{F_1}+0{\cdot}159M\frac{L}{F_2}$	$1{\cdot}043M\frac{L}{F_1}+0{\cdot}200M\frac{L}{F_2}$	$0{\cdot}731M\frac{L}{F_1}-0{\cdot}110M\frac{L}{F_2}$	$0{\cdot}807M\frac{L}{F_1}+0{\cdot}013M\frac{L}{F_2}$	$0{\cdot}734M\frac{L}{F_1}-0{\cdot}061M\frac{L}{F_2}$
1·0	$0{\cdot}706M\frac{L}{F_1}+0{\cdot}118M\frac{L}{F_2}$	$0{\cdot}904M\frac{L}{F_1}+0{\cdot}151M\frac{L}{F_2}$	$0{\cdot}626M\frac{L}{F_1}-0{\cdot}128M\frac{L}{F_2}$	$0{\cdot}720M\frac{L}{F_1}+0{\cdot}016M\frac{L}{F_2}$	$0{\cdot}626M\frac{L}{F_1}-0{\cdot}078M\frac{L}{F_2}$
2·0	$0{\cdot}516M\frac{L}{F_1}+0{\cdot}065M\frac{L}{F_2}$	$0{\cdot}687M\frac{L}{F_1}+0{\cdot}086M\frac{L}{F_2}$	$0{\cdot}467M\frac{L}{F_1}-0{\cdot}134M\frac{L}{F_2}$	$0{\cdot}570M\frac{L}{F_1}+0{\cdot}015M\frac{L}{F_2}$	$0{\cdot}462M\frac{L}{F_1}-0{\cdot}092M\frac{L}{F_2}$
4·0	$0{\cdot}338M\frac{L}{F_1}+0{\cdot}028M\frac{L}{F_2}$	$0{\cdot}468M\frac{L}{F_1}+0{\cdot}039M\frac{L}{F_2}$	$0{\cdot}314M\frac{L}{F_1}-0{\cdot}114M\frac{L}{F_2}$	$0{\cdot}407M\frac{L}{F_1}+0{\cdot}011M\frac{L}{F_2}$	$0{\cdot}308M\frac{L}{F_1}-0{\cdot}088M\frac{L}{F_2}$
8·0	$0{\cdot}201M\frac{L}{F_1}+0{\cdot}010M\frac{L}{F_2}$	$0{\cdot}287M\frac{L}{F_1}+0{\cdot}014M\frac{L}{F_2}$	$0{\cdot}192M\frac{L}{F_1}-0{\cdot}081M\frac{L}{F_2}$	$0{\cdot}261M\frac{L}{F_1}+0{\cdot}006M\frac{L}{F_2}$	$0{\cdot}188M\frac{L}{F_1}-0{\cdot}068M\frac{L}{F_2}$

ends B_1 of these beams subjected to the dead load. Thus when, for instance, the ratio of the sum of the stiffnesses is 2, the total moment applied to the stanchion at B_1 due to a dead load of intensity w on the beams is equal to

$$0{\cdot}601 \times \tfrac{1}{12}\, w(l_1^2 - l_2^2)$$

where l_1 and l_2 are the lengths of the beams.

In the same way the total moment applied to the stanchion at B_1, when the live load is arranged on the beams (1) so that the stanchion length B_0B_1 bends in "double curvature" (*Figs. 27* (*d*)) and (2) so that it bends in "single curvature" (*Figs. 27* (*f*)), can be written down from Table VIII in terms of the live loads on beams A_1B_1 and B_1C_1. These moments are (1) $0{\cdot}687\ M^L_{F_1} + 0{\cdot}086\ M^L_{F_2}$ and (2) $0{\cdot}467\ M^L_{F_1} - 0{\cdot}134\ M^L_{F_2}$; $M^L_{F_1}$ and $M^L_{F_2}$ being the "fixed-end moments" at the ends B_1 of the beams subjected to live load. The end moments in the stanchion-lengths are found by dividing these total moments between the upper and lower stanchion-lengths in proportion to their stiffnesses. It will be realized from a study of the tests on existing buildings that this partition of the total moment is not strictly true, but it leads to no serious error in the case of clothed frames and no better approximation can be found in the existing state of knowledge.

Although in compiling the Tables, such as Tables VII and VIII, to give safe values of the moments in any frame it was necessary to assume the proportions and layout of all members (other than those defined in the ratio $\left(\dfrac{K_{BR} + K_{BL}}{K_U + K_L}\right)$) to be such that the greatest possible moments would be developed in the stanchion, an examination of the moments in particular frames showed that the use of the Tables does not lead to undue extravagance except in certain top stanchion-lengths, where an overestimate of as much as 27 per cent. has been found.

Stresses in a Stanchion.

The reactions at the ends of the stanchion-length are now known, and the next step is to find a convenient way of checking the suitability of the section chosen for the member. This entails finding the maximum total stress in the member or, more conveniently, demonstrating that the maximum stress does not exceed a certain permissible value. It is not difficult to derive an expression showing what must be the values of the end bending stresses (f_A and f_B) arising from end moments M_A and M_B (*Fig. 31*, p. 192) acting on a strut AB, subjected also to a given end load P, in the plane in which buckling

would occur, if the total maximum stress is not to exceed a certain permissible value p'.

This expression is

$$\frac{p'-p+f'}{f_A+f'}=\sin\alpha x\cos\alpha L\left(\frac{f_B+f'}{f_A+f'}+\sin\alpha L\cot\alpha x-\cos\alpha L\right), \quad (1)$$

the value of x being given by

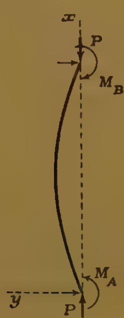
$$\tan\alpha x=\operatorname{cosec}\alpha L\left(\frac{f_B+f'}{f_A+f'}-\cos\alpha L\right), \quad . \quad . \quad . \quad (2)$$

where α denotes $\sqrt{\frac{P}{EI}}$,

I	,,	the minimum moment of inertia of the section,
p	,,	P/A, A being the cross-sectional area,
L	,,	the length of the member, and
f'	,,	a constant, representing the imperfections of the member.

Although these equations are too complex for use in design, it is a simple matter to present the results obtained from them in the form

Fig. 31.



of families of curves showing, for any ratio of f_B/f_A, and therefore of M_B/M_A, the value the maximum end bending stress, f_A, can have without raising the total maximum stress in the member above the permissible value p'. When M_B and M_A are known, therefore (from Tables VII and VIII), together with P, the axial load, which is easily estimated in any case, the suitability of the section can be tested.

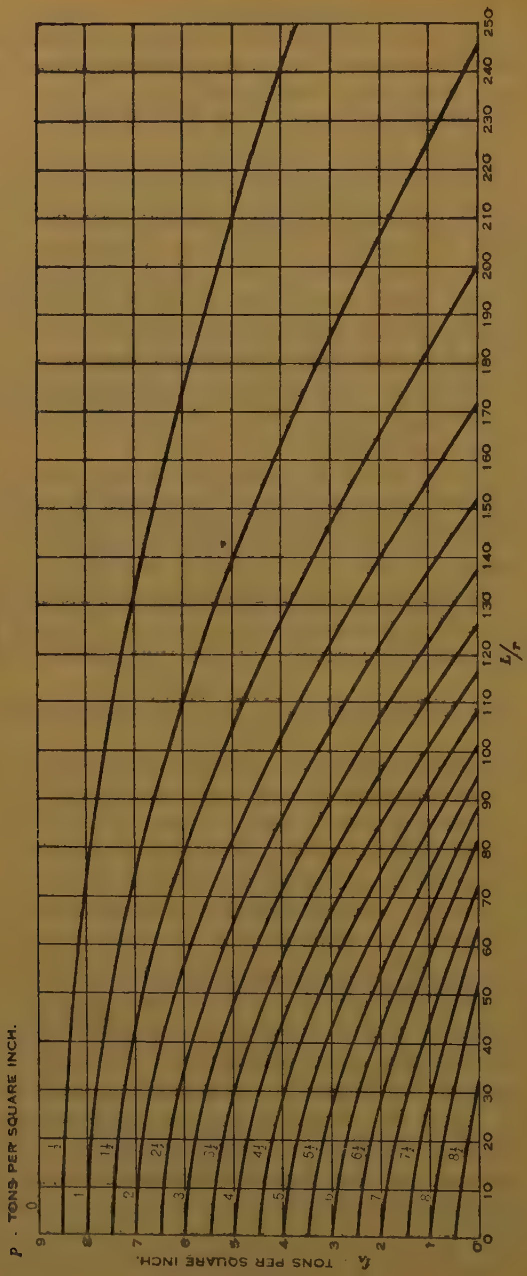
This method of testing is complicated by the necessity of determining the ratio M_B/M_A, and it was thought worth while to introduce a simplification. This can be done by considering only that moment at the top end of the stanchion-length, which, in the type of frame

under consideration, is always greater than that at the bottom end, and by assigning to the moment at the bottom end a limiting value such that safety is ensured. It is not difficult to see that in the case of "single curvature" bending the limit is given by assuming $M_B/M_A = 1$. When the bending of the member is in "double curvature" the limit is given by $M_B/M_A = -0{\cdot}268$. The relevant information given by the families of curves can now be embodied in only two sets, *Figs. 32* and *33* (pp. 194 and 195), which are based on a load-factor of 2 and a yield-stress in the material of 18 tons per square inch. With these curves, which also make allowance for the effects of imperfections in the stanchions not included in the calculation of the end moments, and with the list of end moments given in Tables VII and VIII, a stanchion can be designed with comparative ease. The method will be most easily appreciated if an intermediate length is considered, the stanchion-lengths above having been already designed.

The first step is the choice of a provisional section for the stanchion-length and the determination of the axial load per unit area (p) arising from the loads applied to the structure. Since the beams will have been designed by the method already described, before a start is made with the stanchion design the stiffnesses of the beams framing into the upper end of the stanchion-length, the fixed end moments, M_{F_1} and M_{F_2}, and the reactions at the ends of these beams, will be known. The stiffness-ratios $\left(\frac{K_{BR}+K_{BL}}{K_U+K_L}\right)$ about both axes of the stanchion can also be set down. These enable the total bending moments on the stanchion due to dead load and to both critical arrangements of live load to be read off from Tables VII and VIII, and it is essential that the ability of the section selected to sustain both these arrangements of load should be checked.

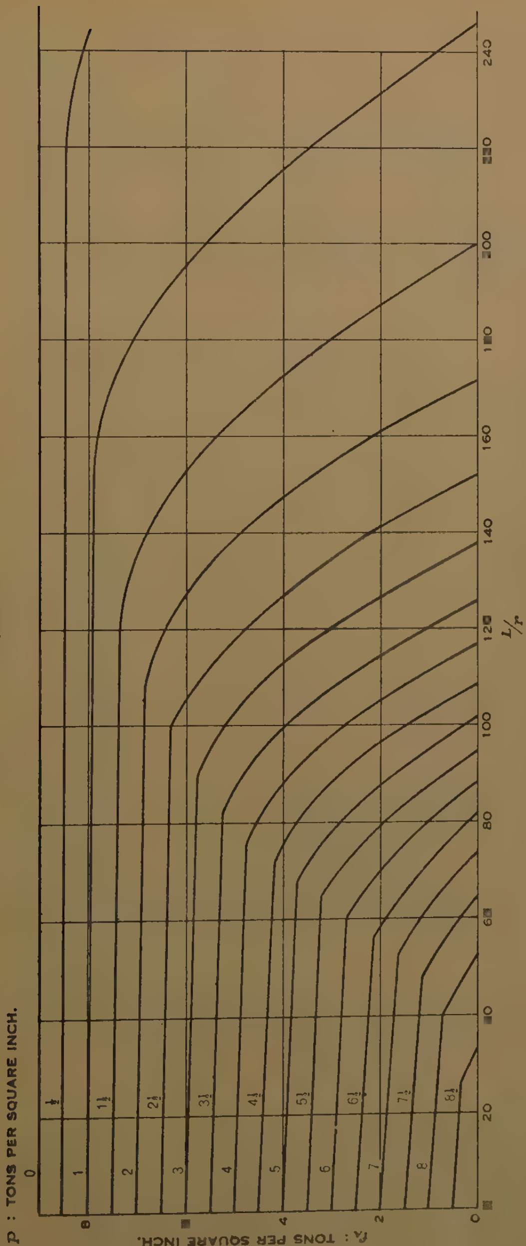
"Single curvature" loading will first be considered. Using the appropriate value of the stiffness-ratio, the total bending moment about the XX axis of the stanchion at the top end due to live load is read from Table VIII (col. 4), and to this must be added the dead-load total bending moment read from Table VII (col. 3). The dead load will actually cause bending of the stanchion-length in "double curvature," but if serious complications are to be avoided some economy must be sacrificed, and the dead-load moment be assumed to produce bending in "single curvature." The sum of these total moments is now divided between the stanchion-length under consideration and that above, in the ratio of their relevant stiffnesses, thus giving the end moment in "single curvature" about the XX axis at the top end of the stanchion-length. The

Fig. 32.



PERMISSIBLE END BENDING STRESS WITH "SINGLE CURVATURE."

Fig. 33.



Permissible End Bending Stress with "Double Curvature."

maximum fibre-stress due to this end moment is then set down. In the same way the maximum fibre-stress due to that arrangement of load producing bending in " single curvature " about the YY axis of the stanchion is set down. The sum of these fibre-stresses can then be taken as the maximum " single curvature " end bending stress in the stanchion-length. If this stress is less than the permissible end bending stress (f_A) given in *Fig. 32* for the stanchion-length, as defined by its slenderness-ratio (L/r) and axial load per unit area (p), then the section selected will be adequate to carry the forces imposed on it when the loads are such as to produce bending in " single curvature."

The suitability of the section to resist " double curvature " bending is checked in a similar way. The sum of the total bending moments about the XX axis of the stanchion at its top end due to live loads (Table VIII, col. 3) and dead loads (Table VII, col. 3), is, as before, divided between the stanchion-lengths in proportion to their relevant stiffnesses, thus giving the bending moment and hence the bending stress in " double curvature " about the XX axis at the top end of the stanchion-length. In the same way the bending stress, due to " double curvature " loading, about the YY axis of the stanchion, is set down. The sum of these stresses can then be taken as the maximum " double curvature " end bending stress, and the suitability of the section checked, as before, from *Fig. 33*.

It might be thought that two other combinations of load, producing " single curvature " bending about one axis and " double curvature " about the other, should be considered. A comprehensive investigation, in which the true maximum stress in the stanchion under such loading has been found in a number of particular cases, makes it appear that the pure " double curvature " or " single curvature " cases dealt with above will always be critical.

The fact that the end bending stresses about both principal axes are added in the above needs some explanation. Expressions (1) and (2), p. 192, refer only to bending in the plane in which buckling would occur, but it can be proved that if the end stresses arising from bending in a plane at right angles are assumed to arise from bending in the plane in which buckling would occur, safety is ensured.

Final Simplified Method.

The Committee's object would not have been attained if the final method suggested for proportioning a stanchion, however sound its basis, had been too complex to be used conveniently in the design-office. A number of practising engineers were good enough to give the First Simplified Method, just described, a trial in their offices.

They were, unfortunately, unanimously of the opinion that while designers had no difficulty in applying the method, the time taken in proportioning a stanchion was prohibitive. The reason was that the two critical loading-conditions had to be considered separately for each stanchion-length. To produce a method acceptable to the designer, further simplification was essential.

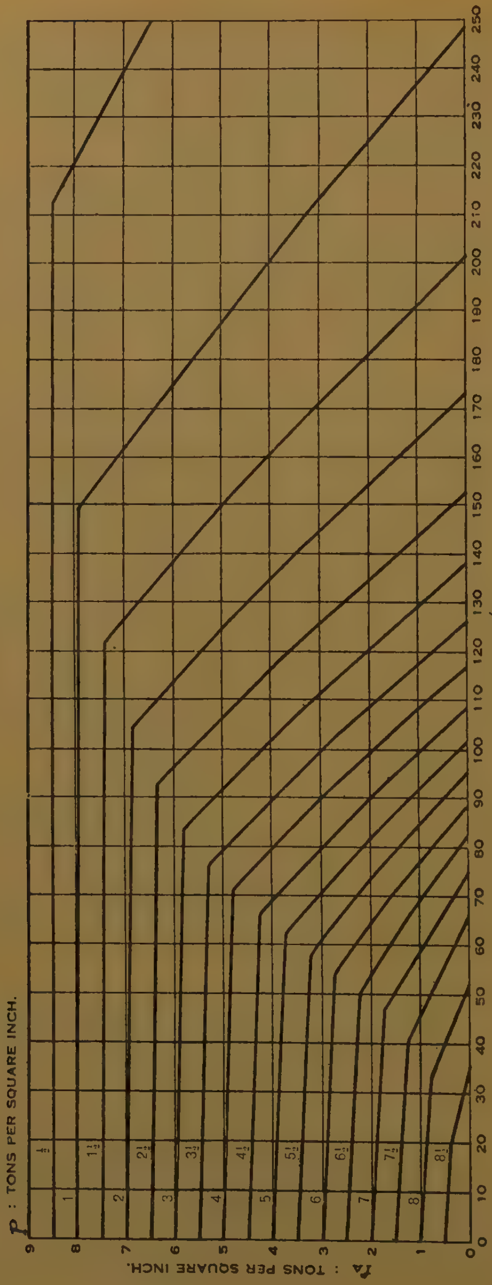
It was necessary to arrange the data so that, while the principle of the original method would not be undermined, only one loading-condition would, in effect, have to be considered. It was, fortunately, found possible, without any serious sacrifice of economy, to prepare one set of curves giving the permissible end bending stress, which would ensure safety whether bending was in " single " or " double " curvature.

For a symmetrical frame in which $M_{F_2} = M_{F_1}$, the bending moment in " single curvature " is never greater than 1/1·7 times the moment in " double curvature." If, therefore, the ordinates of the " single-curvature " curve (*Fig. 32*) for any value of p are multiplied by 1·7, then the composite lower-limit curve, formed from this and the corresponding " double-curvature " curve (*Fig. 33*) by taking that portion of each which gives the lower value of the permissible end bending stress (f_A), will, if used in conjunction with the " double-curvature" moments, ensure safety in all cases, whether the critical loading is such as to produce bending in " double " or " single " curvature. Such composite lower-limit curves are shown in *Fig. 34* (p. 198).

These composite strut-curves are so drawn that safety is ensured provided that the moment in " single curvature " is not greater than 1/1·7 times the moment in " double curvature." It must be remembered that the curve giving the permissible end bending stress (f_A) in " single curvature " is based on the assumption that the bending moments at the ends of the stanchion-length are equal. This is important, since as a result, although in an unsymmetrical frame the bending moment for " single-curvature " loading may be more than 1/1·7 times that due to " double curvature," the composite strut-curves are always satisfactory.

Some further consideration must now be given to the moments of Tables VII and VIII. It is not easy to write down the axial load in a stanchion-length resulting from the arrangement of load on the beams which produces these moments, but it is known to be less than that arising from all beams loaded. The conditions arising when a stanchion-length is subjected to this full axial load, together with the end moments deduced from Table VII or VIII, are therefore more rigorous than the real worst conditions. The combination of circumstances giving rise to the latter is likely to arise but rarely in practice, so that if the rather worse conditions,

Fig. 34.



PERMISSIBLE END BENDING STRESS (COMPOSITE CURVE).

full axial load and full end moments, are assumed, a satisfactory stanchion-section could be produced if the load-factor chosen was less than 2 (say 1·25). The stanchion, while being safe under these impossibly rigorous conditions, would have a greater load-factor than 1·25 under more normal loads. Unfortunately it was considered inadvisable to set out the method of design in this way; instead, it was decided to retain the load-factor of 2, used in plotting the curves in *Figs. 32, 33* and *34*, but to follow the lead set by existing codes, Clause 35 (*e*), p. 129, and to reduce the live axial load assumed in all storeys below the topmost. If this reduction is justifiable in the live axial load, some similar reduction is justifiable in the end moments due to the live load. This was arranged by omitting all the terms in M_{F_2} from Table VIII, thus giving a reduction which varies with the stiffness-ratio but which never exceeds 20 per cent. It must be realized that this omission of the M_{F_2} term is merely a device to secure a reduction in the total moment, while at the same time simplifying the calculation of that moment. The omission

TABLE IX.—TOTAL BENDING MOMENT DUE TO DEAD LOAD.

$\frac{K_{BR} + K_{BL}}{K_U + K_L}$,	Bottom length; $\left(\frac{\text{total moment}}{M_F^D}\right)$.	Intermediate length; $\left(\frac{\text{total moment}}{M_F^D}\right)$.	Topmost length $\left(\frac{\text{total moment}}{M_F^D}\right)$.
0	0·862	1·000	1·000
0·1	0·820	0·975	0·957
0·2	0·783	0·950	0·918
0·3	0·752	0·920	0·882
0·4	0·725	0·893	0·852
0·5	0·697	0·865	0·822
0·6	0·671	0·842	0·794
0·7	0·648	0·820	0·769
0·8	0·627	0·798	0·745
0·9	0·606	0·776	0·722
1·0	0·589	0·754	0·704
1·25	0·545	0·709	0·659
1·5	0·511	0·669	0·619
1·75	0·482	0·632	0·583
2·0	0·452	0·601	0·555
2·5	0·405	0·546	0·503
3·0	0·367	0·500	0·461
3·5	0·336	0·462	0·426
4·0	0·310	0·429	0·396
5·0	0·268	0·375	0·349
6·0	0·236	0·333	0·310
7·0	0·211	0·300	0·279
8·0	0·191	0·273	0·256
10·0	—	—	0·227
12·0	—	—	0·197
14·0	—	—	0·174
16·0	—	—	0·156

does not mean that the critical arrangements of load, which form the whole basis of this work, have in any way been altered.

The moments in topmost lengths of internal stanchions and in external stanchions needed special treatment, which will not be gone into here, but it was not difficult to produce a table of amended moments suitably reduced which, if used in conjunction with the composite strut-curve, brings about a very considerable saving of labour as compared with the first method described. The amended moments are given in Tables IX and X.

TABLE X.—TOTAL BENDING MOMENT DUE TO LIVE LOAD.

$\frac{K_{BR} + K_{BL}}{K_U + K_L}$.	Bottom length; $\left(\frac{\text{total moment}}{M_F^L}\right)$.	Intermediate length; $\left(\frac{\text{total moment}}{M_F^L}\right)$.	Topmost length; $\left(\text{total moment}/M_F^L\right)$. Internal stanchion.	Topmost length; $\left(\text{total moment}/M_F^L\right)$. External stanchion.
0	1·149	1·359	1·360	1·000
0·1	1·080	1·290	1·261	0·961
0·2	1·017	1·230	1·175	0·925
0·3	0·961	1·175	1·098	0·892
0·4	0·912	1·125	1·031	0·861
0·5	0·871	1·082	0·968	0·834
0·6	0·834	1·043	0·915	0·807
0·7	0·796	1·004	0·869	0·781
0·8	0·763	0·968	0·823	0·759
0·9	0·732	0·934	0·782	0·738
1·0	0·706	0·904	0·746	0·720
1·25	0·648	0·837	0·669	0·673
1·5	0·596	0·780	0·602	0·635
1·75	0·554	0·729	0·548	0·601
2·0	0·516	0·687	0·503	0·570
2·5	0·456	0·615	0·431	0·517
3·0	0·408	0·556	0·390	0·474
3·5	0·370	0·508	0·358	0·436
4·0	0·338	0·468	0·334	0·407
5·0	0·289	0·404	0·293	0·357
6·0	0·252	0·356	0·261	0·318
7·0	0·224	0·318	0·235	0·287
8·0	0·210	0·287	0·214	0·261
10·0	—	—	0·181	0·222
12·0	—	—	0·157	0·193
14·0	—	—	0·139	0·171
16·0	—	—	0·124	0·153

In addition to what has been described, provision had to be made for designing frames of all types. The Tables given above could only be compiled in their simple form on the assumption that all stanchion-lengths in the same storey were of the same stiffness. It was found possible to give correction-factors for use when this condition did not exist, by which the tabulated moments could be multiplied to give

safe values of the moments in any unsymmetrical frame (Clause 16 below). A further correction-factor was needed when the intensity of load on the beam framing into the lower end of a stanchion-length was greater than that on the beam framing into the upper end (Clause 15 below).

The more important clauses setting out the method are :—

(10) For each stanchion length a steel section shall be chosen and its adequacy shall be tested by the method set out in Clauses 11–16.

(11) The total load (P) carried by each stanchion length shall be determined on the assumption that all beams are freely hinged at their ends.

(12) For the purpose of calculating the total load the live load for the roof and topmost storey shall be calculated in full, but for the lower storeys a reduction of the live loads may be allowed in accordance with the following Table :—

Next storey below topmost storey	10 per cent. reduction of live load.
Next storey below	20 per cent. reduction of live load.
Next storey below	30 per cent. reduction of live load.
Next storey below	40 per cent. reduction of live load.
All succeeding storeys	50 per cent. reduction of live load.

No such reduction shall be allowed on any floor scheduled for an applied loading of 100 lb. or more per square foot.

(13) The axial load in tons per square inch of gross cross-sectional area of steel (p) shall be determined.

(14) Where the fixed end moments of the beams framing into the upper and lower ends of a stanchion length are the same in each bay and where each of these beams is attached at its remote end to a stanchion having a relevant stiffness at least as great as that of the stanchion under consideration, the procedure shall be as follows :—

(1) M^D_{FX}, the moment about the XX axis of the stanchion due to dead load on the beams attached to the upper end of the stanchion length, shall be calculated. M^D_{FX} shall be the algebraic sum of the resolved components, about the XX axis of the stanchion, of the fixed end moments at the ends, attached to the stanchion, of these beams and of the moments about the XX axis of the stanchion due to the reactions at their ends. The total moment on the stanchion about its XX axis at the upper end of the stanchion length

due to the dead load shall then be determined from Table II for the appropriate value of the stiffness ratio $\frac{K_{BR} + K_{BL}}{K_U + K_L}$, where K_L and K_U are the relevant stiffnesses, that is in this case about the XX axis, of the stanchion length under consideration and of the one above, and K_{BR} and K_{BL} are the relevant stiffnesses of the beams, that is in this case the stiffnesses of the beams framing into the upper end of the stanchion length at right angles to the XX axis of the stanchion.

(2) In the same way the maximum moment M^L_{FX} about the XX axis of the stanchion due to the most unfavourable arrangement of live load on the beams attached to the upper end of the stanchion length shall be calculated. The total moment on the stanchion about its XX axis at the upper end of the stanchion length due to live load shall then be determined from Table III.

(3) The end bending moment in the stanchion length about its XX axis shall be taken as the sum of the dead and live load total moments determined in Clause 14 (1)–(2) multiplied by the ratio of the relevant stiffness of the stanchion length under consideration to the sum of the relevant stiffnesses of that stanchion length and of the one above. The maximum bending stress in the steel section at the upper end of the stanchion length due to this end bending moment shall be calculated.

(4) The same procedure as that set out in Clause 14 (1)–(3) shall be adopted for the determination of the maximum end bending stress due to the moments about the YY axis.

(5) Where a beam makes a skew connection to a stanchion the fixed end moment at the end of the beam shall, for the determination of M^D_F and M^L_F, be resolved into its components about the XX and YY axes of the stanchion, and the relevant stiffnesses of the beam shall be the stiffness of the beam multiplied by the sines of the angles between the axis of the beam and the XX and YY axes of the stanchion respectively.

(6) The total end bending stress shall be taken as the sum of the maximum end bending stresses calculated under Clause 14 (1)–(5). The magnitude of this

total end bending stress shall not exceed the value given for the stanchion length, as defined by its slenderness ratio (L/r_y), by the curve of *Fig. 34* appropriate to the axial load per unit area (p).

15. Where the fixed end moment of a beam framing into the lower end of a stanchion length is greater than the fixed end moment of the corresponding beam framing into the upper end, then the fixed end moment of the upper beam shall be increased in the ratio $\frac{2+V}{3}$, where V is the ratio of the fixed end moment of the lower beam to that of the upper. This increased moment shall be used in the determination of the total end bending stress at the upper end of the stanchion length. The total end bending stress at the lower end of the stanchion length shall also be determined using the actual values of the fixed end moments of the beams framing into the lower end of the stanchion length. The greater of these two end bending stresses shall then be taken as the total end bending stress to be used in determining the adequacy of the section as in Clause 14 (6).

16. (1) Where a beam frames at its remote end into the web of another beam, then the fixed end moment at the end of the beam attached to the stanchion shall be increased by the appropriate correction factor as given in Table XI.

TABLE XI.—CORRECTION FACTOR (CONNECTION TO BEAM AT REMOTE END).

$\frac{K_{BR}+K_{BL}}{K_U+K_L}$.	
0·1	1·47
0·5	1·36
1·0	1·29
2·0	1·20
4·0	1·13
8·0	1·07
16·0	1·04

(2) Where a beam frames at its remote end into a stanchion of less stiffness than the stanchion under consideration, then the fixed end moment shall be increased by the appropriate correction factor (Table XII, p. 204). Where the

TABLE XII.—CORRECTION-FACTOR (CONNECTION TO STANCHION AT REMOTE END).

$\frac{K_{BR} + K_{BL}}{K_U + K_L}$.	Stiffness ratio, S.			
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{1}{2}$
0·1	1·27	1·18	1·10	1·04
0·5	1·30	1·25	1·18	1·09
1·0	1·25	1·22	1·17	1·09
2·0	1·18	1·16	1·14	1·08
4·0	1·11	1·10	1·09	1·05
8·0	1·07	1·06	1·05	1·03
16·0	1·04	1·03	1·03	1·02

$$S = \frac{\text{sum of stiffnesses of upper and lower stanchion lengths at remote end of beam}}{\text{sum of stiffnesses of upper and lower stanchion lengths at end under consideration}}.$$

stiffness of the stanchion at the remote end is not less than one-half that of the stanchion under consideration no correction factor need be used. Where the beam frames into the flange of the stanchion under consideration and is attached at its remote end to the web of another stanchion, the section of which is not known, the ratio (S) shall be taken as $\frac{1}{16}$.

EFFECT OF WIND-LOADS.

Before proceeding to a worked example, attention must be drawn to the decision made by the Committee that the method of design finally recommended should apply "to the steel framework of building structures formed with horizontal girders and vertical columns and provides only for the stresses caused by vertical forces. Any structure for which the method of design is used must be so constructed as to resist horizontal forces due to wind or other causes, without significant horizontal sway, by means of its floor slabs in association with vertical walls or braced vertical frames."

This decision was based on the evidence provided by a considerable investigation of the effect of wind-loads on frames having semi-rigid connections. It was found that the bending moments in the stanchions due to wind-loads increased rapidly in magnitude as the connections departed from the condition of complete rigidity. This tendency is shown in the tests on the experimental frame, p. 136. It was considered desirable to relieve such frames of the responsibility for the wind-shear and therefore, where floors and walls do not supply adequate bracing, special wind-bents are demanded.

The investigation showed also that the assumptions usually made to-day when wind-loads are taken into account are not satisfactory

for use in the design of these special bents. Further investigation of the problem is desirable, but it has been possible, from the data collected for the Committee, to suggest a method which is simple to apply and which, it is believed, is an improvement on those commonly used.

Application of the Method of Design.

The most satisfactory way of showing the ease with which the Committee's method of design can be applied is to give detailed calculations for a particular case.

The floor plan of a six-storey building is shown in *Figs. 35* (p. 206), together with details of the loads assumed. Beams Nos. 21, 92 and 102 at a floor-level, and stanchion No. 10, into which they frame, will be designed.

Table XIII (facing p. 208) shows the beam-calculations. In the first column is placed the beam-mark, and in the second the span of the beam between supporting faces. The next four columns refer to the left-hand end of the beam, giving the restraining moment, class of connection, fixed end moment and reaction, respectively. In the seventh column are the details of the loads. The next four columns refer to the right-hand end of the beam.

Calculations for Beam No. 21.—The first step, the loads having been entered in col. 7, is to set down in cols. 6 and 8 the reactions at the left- and right-hand ends of the beam. The fixed end moments are then entered in cols. 5 and 9. As, in this case, the load is uniformly distributed, the fixed end moments due to live load are each

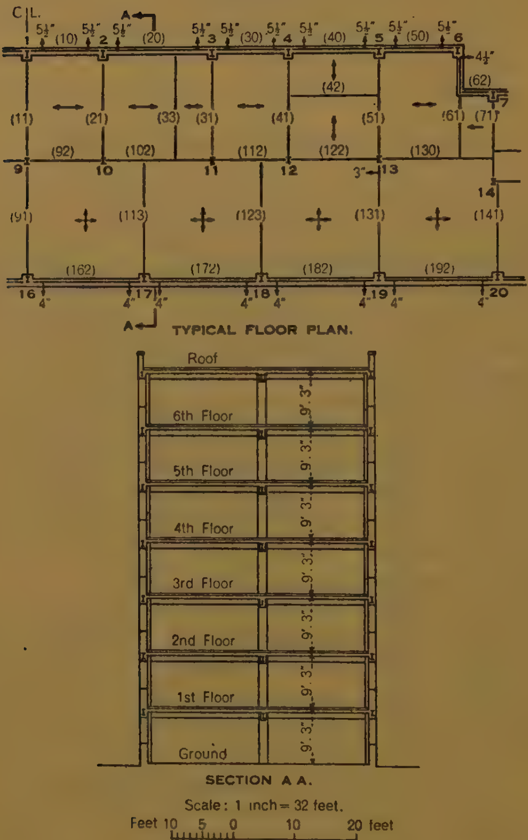
$$\tfrac{1}{12} \times 3{\cdot}8 \times 16{\cdot}5 \times 12 = 62{\cdot}5 \text{ ton-inches},$$

and due to dead load

$$\tfrac{1}{12} \times 8{\cdot}8 \times 16{\cdot}5 \times 12 = 145 \text{ ton-inches}.$$

Beam No. 21 is to be connected at its ends to the flanges of stanchions by connections of class "B" (6-inch by 4-inch by ½-inch flange-cleats), and a provisional beam-depth "D" must be chosen so that the restraining moments may be estimated. In this case "D" was taken as 12 inches, so that in col. 12 the value of $M_F/2D$ can be entered, M_F being the mean of the total fixed end moments at the ends of the beam, from cols. 5 and 9. All the necessary information is now available for the evaluation of the restraining moments at the ends of the beam from the appropriate curve of *Fig. 29* (*a*), which refers to connections of class "B." Since the length of the

Figs. 35.



Loading Data.

Roof:

Garden superimposed	40	lbs. per square foot $= \frac{1}{56}$ ton.
Constructional slab	45	,, ,, ,,
Screed	12	,, ,, ,,
Asphalt	10	,, ,, ,,
Flower-boxes	10	,, ,, ,,
Ceiling	5	,, ,, ,,
	82	,, ,, ,, $= \frac{1}{27 \cdot 2}$ ton.

[*Continued at foot of next page.*

beam is 16·5 feet and $M_F/2D$ is 8·6 tons, then M/D is 4·6 tons and the restraining moment (M) at each end of the beam is

$$4{\cdot}6 \times 12 = 55{\cdot}2 \text{ tons-inches.}$$

The maximum bending moment in the beam is calculated in col. 13, due allowance being made for the restraining moment M, and, taking 9 tons per square inch as the permissible stress, the section-modulus required is found to be 28·6 inch³ units. The beam chosen to fulfil this requirement must have a depth of 12 inches, and it will be found that a 12-inch by 5-inch by 30-lb. joist is satisfactory.

Calculations for Beam No. 102.—Beam No. 102 is connected to the webs of the stanchions. According to Clause 3 (2) (p. 187), no allowance may be made for the restraining moment at the end of such a member except in the case where the connection at the end is balanced by a connection of the same class to a beam on the opposite side of the member, with rivets or bolts serving both connections. As it is clear that the economical sections of beams Nos. 92, 102 and 112 will be very different, the connections at their ends will not be served by common rivets or bolts. No allowance has, therefore, been made in Table XIII for restraining moments at the ends of beam No. 102. The fixed end moments have, however, been entered in cols. 5 and 9, since they will be needed in the stanchion-calculations.

In dealing with a beam of this type allowance may be made for restraining moment at the end if special provision is made to stiffen the web of the stanchion by some means, such as arranging the beam-depths so that the connections on the two sides of the stanchion-web are served by common rivets or bolts.

Calculations for Beam No. 92.—Beam No. 92 is also attached to stanchion-webs. It is different from beam No. 102 in that allowance could, in theory, be made for a restraining moment at the end framing into stanchion No. 9, since on the other side of the stanchion-web is an exactly similar beam. The magnitude of the restraining moment is, however, too small to be taken into account.

Domestic floors :

Superimposed	40 lbs. per square foot = $\frac{1}{56}$ ton.
Constructional slab	45 ,, ,, ,,
Finish (boards and battens) . .	5 ,, ,, ,,
Ceiling	5 ,, ,, ,,
Partitions	27 ,, ,, ,,
	82 ,, ,, ,, = $\frac{1}{27\cdot[illegible]}$ ton.

Walls :

Main frontage	13½-inch brick = $\frac{1}{16}$ ton.
Garden court	9-inch brick = $\frac{1}{23\cdot5}$ ton.

Calculations for Stanchion No. 10.—The arrangement of the stanchion calculations (Table XIV, facing p. 209), like that of the beam calculations, has been made to conform as nearly as possible to existing practice.

Normally, the same section will be used for the two topmost lengths of a stanchion. In Table XIV this section has been chosen from a consideration of the conditions in the stanchion-length next below the topmost length. In certain cases more rigorous conditions may occur in the topmost length, and the adequacy of the section there should be tested. The first step is to set down on the loading-diagram the reactions and fixed end moments due to both live and dead load for each beam framing into the stanchion. In col. 4 the loads coming on to the stanchion are set out. In the cases illustrated these loads are made up of the weight of the stanchion and casing and of the beam reactions, since for simplicity in the beam calculations the load carried by each was assumed to be that due to the full area of floor between stanchion centres. The load carried by a beam may, however, be taken as due to that part of the floor between stanchion-flanges, and where use is made of this refinement the loads entered in col. 4, which must give the total load coming on to the stanchion, will be greater than the reactions from the beams. An improvement might also be made by providing two columns for loads, one for dead load and the other for live load. This would enable the permitted reduction in the latter to be made with ease.

A stanchion-section (in the case illustrated in Table XIV, an I-section, 10 inches by 6 inches by 40 lb., plus two 9-inch by $\frac{5}{8}$-inch flange-plates) is now chosen and the axial load per square inch of cross section, and the slenderness-ratio, are entered in col. 5. These being known, the permissible end bending stress (f_A) may now be read off from the appropriate curve (*Fig. 34*) and entered in the table. This is a convenient time for calculating, in col. 8, the stiffness-ratios $\dfrac{K_{BR}+K_{BL}}{K_U+K_L}$ about both the XX and YY axes of the stanchion. The next step is the calculation (col. 6) of the moments about the principal axes of the stanchion, due to the reactions and fixed end moments entered on the loading-diagram. In the case illustrated in Table XIV the moment due to dead load about the XX axis is made up as follows: 145 tons-inches (the dead-load fixed-end moment of beam No. 21), plus $4{\cdot}4 \times 5{\cdot}4$ tons-inches (the dead-load reaction at the end of beam No. 21, multiplied by the distance of that reaction from the XX axis of the stanchion). The live-load moment about the XX axis is similarly $(62{\cdot}5 + 1{\cdot}9 \times 5{\cdot}4)$ tons-inches.

The moments about the YY axis are calculated in the same way. Since beams Nos. 92 and 102 are connected to the web of the stanchion,

Table XIII.—Beam Sheet.

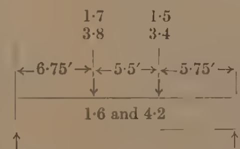
Col. 1	2	3	4	5	6	7	8	9	10	11	12	13	14
		Left hand.					Right hand.						
Beam-mark.	Span.	Restraining moment.	Class of connection.	Fixed end moment: tons-inches.	Reaction: tons.	Details of loading.	Reaction: tons.	Fixed end moment: tons-inches.	Class of connection.	Restraining moment: tons-inches.	$\frac{M_F}{2D}$	Calculations for maximum bending moment.	Section selected.
	ft. ins.												
102	18 0	—	Web connection, no restraint.	53·5 22·4 28·8 104·7 120·0 51·0 — 75·6 246·6	1·05 0·5 0·8 2·4 1·1 — 2·1 7·95	*Live Load* From 113 . . . . . . 1·7 From 33 . . . . . . 1·5 Floor . . . . 1·6 *Dead Load* From 113 . . . . . . 3·8 From 33 . . . . . . 3·4 Floor . . . . 3·2 B. and C. . . 1·0 	0·65 1·0 0·8 1·4 2·3 — 2·1 8·25	32·3 47·8 28·8 108·9 72·1 108·4 — 75·6 256·1	Web connection, no restraint.	—	—	Maximum bending moment at— $\frac{7\cdot95 - 5\cdot5}{0\cdot322} = 7\cdot6$ feet from R_L. Bending moment $= 7\cdot95 \times 7\cdot6 = 60\cdot5$ $- 5\cdot5 \times 0\cdot85 = 4\cdot7$ $- 2\cdot45 \times 3\cdot8 = 9\cdot3$ 14·0 14·0 46·5 tons-feet Section modulus required = 69·75 inch³ units.	I-section, 16 inches × 6 inches × 50 lb. $K = 34\cdot4$
92	12 6	—	Web connection, no restraint.	13·8 — 36·3	0·55 — 1·45 2·00	*Live Load* Floor . . . . 1·1 *Dead Load* Floor . . . . 2·3 B. and C. . 0·6	0·55 — 1·45 2·00	13·8 — 36·3	Web connection, no restraint.	—	—	Maximum bending moment $= \frac{4 \times 12\cdot5 \times 12}{8} = 75$ tons-inches. Section modulus required = 9·4 inch³ units.	I-section, 7 inches × 3½ inches × 15 lb. $K = 2\cdot9$
21	16 6	— — 55·2	Flange connection. Class "B"	62·5 — 145	1·9 — 4·4 6·3	*Live Load* Floor . . . . 3·8 *Dead Load* Floor . . . . 7·9 B. and C. . 0·9	1·9 — 4·4 6·3	62·5 — 145	Flange connection. Class "B"	— — — 55·2	$\frac{207\cdot5}{24} = 8\cdot6$	Free bending moment $= \frac{12\cdot6 \times 16\cdot5 \times 12}{8} = 312$ tons-inches. Maximum bending moment $= 312 - 55\cdot2 = 257$ tons-inches. Section modulus required $= \frac{257}{9} = 28\cdot6$ inch³ units.	I-section, 12 inches × 5 inches × 30 lb. $K = 12\cdot5$

TABLE XIV.—STANCHION SHEET.

Col. 1	2	3	4	5		6	7		8
Floor.	Section.	Loading diagram.	Loads.	Axial load per square inch and allowable bending stress.		Fixed-end moment to be used in stanchion-design.	Actual bending stress. $f=\frac{1}{Z}\times\frac{K_L}{K_U+K_L}\times[C^D\times M_F^D+C^L\times M_F^L]$.		$\frac{K_{BR}+K_{BL}}{K_U+K_L}$.
R	9 feet 3 inches.				XX				
			6·3 7·9 2·0 0·8		YY				
6			17·0						
	9 feet 3 inches. I-section 10 inches × 6 inches × 40 lb. Two 10 inches × ½ inch plates.	21 $R^D=$ 4·4 $R^L=$ 1·9 $M_F^D=$145 $M_F^L=$ 62·5 $R^D=1·45$ | $R^D=5·6$ $R^L=0·55$ | $R^L=2·35$ 92 — 102 $M_F^D=36·3$ | $M_F^D=246·6$ $M_F^L=13·8$ | $M_F^L=104·7$		$p=\frac{34·1}{21·77}$ $=1·57$ $l/r=\frac{111}{2·19}$ $=50$	XX	D, 145+4·4×5·5 =169 L, 62·5+1·9×5·5 =73	$f=\frac{1}{87·3}\times\frac{1}{2}[0·97\times169+1·28\times73]$ $=\frac{164+93}{174·6}$	=1·47	$\frac{12·5}{2\times51·9}$ =0·12
			6·3 7·9 2·0 0·9		YY	D, 246·6−36·3 =210·3 L =104·7	$f=\frac{1}{21}\times\frac{1}{2}[0·65\times210·3+0·75\times104·7]$ $=\frac{137+78}{42}$	=5·12	$\frac{34·4+2·9}{2\times11·4}$
5			34·1	$f_A=7·3$				Total =6·59	=1·63
	9 feet 3 inches.		16·2		XX				
			1·0		YY				
4			51·3						
	9 feet 3 inches.		16·2		XX				
			1·0		YY				
3			68·5						
	9 feet 3 inches. I-section 10 inches × 6 inches × 40 lb. Two 10 inches × ¾ inch plates.		16·2	$p=\frac{85·7}{26·77}$ $=3·2$ $l/r=\frac{111}{2·34}$ $=47·5$	XX	D, 145+4·4×5·75 =170·3 L, 62·5+1·9×5·75 = 73·4	$f=\frac{1}{111}\times\frac{1}{2}[0·98\times170·3+1·30\times73·4]$ $=\frac{167+95·4}{222}$	=1·18	$\frac{12·5}{2\times69}$ =0·09
			1·0		YY	D, 246·6−36·3 =210·3 L =104·7	$f=\frac{1}{29·3}\times\frac{1}{2}[0·72\times210·3+0·85\times104·7]$ $=\frac{152+89}{58·6}$	=4·11	$\frac{37·3}{2\times15·8}$
2			85·7	$f_A=5·7$				Total =5·29	=1·18
	9 feet 3 inches.		16·2		XX				
			1·0		YY				
1			102·9						
	9 feet 3 inches. I-section 10 inches × 6 inches × 40 lb. Two 12 inches × ⅝ inch plates.			$p=\frac{120·1}{26·8}$ $=4·5$ $l/r=\frac{111}{2·74}$ $=41$	XX	D, 145+4·4×5·6 =169·6 L, 62·5+1·9×5·6 = 73	$f=\frac{1}{112}\times\frac{1}{2}[0·83\times169·6+1·1\times73]$ $=\frac{141+80·5}{224}$	=0·99	$\frac{12·5}{2\times68}$ =0·09
			16·2 1·0		YY	D, 246·6−36·3 =210·3 L =104·7	$f=\frac{1}{33·6}\times\frac{1}{2}[0·62\times210·3+0·75\times104·7]$ $=\frac{132+78·5}{67·2}$	=3·14	$\frac{37·3}{2\times21·8}$
G			120·1	$f_A=4·3$				Total =4·13	=0·85

the reactions from these beams may be assumed to produce no moment about the YY axis of the stanchion. The moment due to dead load is then 217 tons-inches (the fixed end moment of beam No. 102), minus 36 tons-inches (the fixed end moment of beam No. 92). Since the maximum live-load moment must be used, that arrangement of live load which gives it must first be found. It will be seen at a glance that the maximum live-load moment about the YY axis occurs when beam No. 102 is loaded and beam No. 92 is unloaded, the live-load moment then being 93 tons-inches.

All the data needed for the determination of the actual total end bending stress in the stanchion are now available, and the method used will be clear from the formula in general terms set out at the top of col. 7. In this formula Z denotes the relevant modulus of section of the stanchion-length, $\frac{K_L}{K_U + K_L}$ is the ratio of the relevant stiffness of the stanchion-length under consideration to the sum of the relevant stiffnesses of that stanchion-length and of the one above, C^D and C^L are the coefficients obtained from the tables of total bending moments due to dead load and due to live load (Tables IX and X), and M_F^D and M_F^L are the moments found in col. 6 due to dead and live load.

The actual total end bending stress is the sum of the end bending stresses about the XX and YY axes. These are calculated separately by the formula, as shown in detail in col. 7. The actual total end bending stress, in this case 6·93 tons per square inch, must be less than f_A, the permissible end bending stress (7·1 tons per square inch), if the stanchion-section chosen is to be adequate.

Conclusion.

As this Paper is nothing more than an outline of the work carried out for the Steel Structures Research Committee no attempt will be made to summarize it.

There is no doubt that the investigations have added much to the store of knowledge of the behaviour of steel structures. The real success of the Committee's main work, the production of a rational method of design, can only be judged by the use which is made of it. Means are now available for basing the design of a steel building-frame on the true behaviour of the structure under the loads which it is its function to sustain. The method puts into the hands of the designer a powerful weapon, enabling him to attack any problem with a confidence which he cannot have when using existing methods.

Unfortunately, under the conditions which usually govern the

erection of building-structures, the case for the immediate adoption of a rational method for the design of the more usual types of riveted or bolted frames is not as strong as it should be. Local authorities lay down regulations for the design of such structures and accept designs which, though without any rational basis, are more cheaply and easily made. As the object of regulating design is to produce sound structures, local authorities should, in their own interests, offer to the designer inducements to use the more exact method now available. Where any departure is made from the usual methods of fabrication or lay-out, such as, for instance, when welding is used to connect the members, it would be unwise to allow the use of any method of design other than that described in the Paper.

Figs. 1, 2, 3, 4, 5, 6, 7, 8, 21, 22 and *25* (Crown copyright reserved) are reproduced from the First and Second Reports of the Steel Structures Research Committee, by permission of the Controller of H.M. Stationery Office.

As already stated, the work described in this Paper was carried out as part of the programme of research of the Steel Structures Research Committee of the Department of Scientific and Industrial Research, and the Author's thanks are due to the Department for permission to publish the Paper. The complete discussion of the results presented will appear in the Final Report of the Committee, which will be published shortly through H.M. Stationery Office.[1]

The Paper is accompanied by twenty-two sheets of drawings and by six photographs, from some of which the Figures and the three pages of half-tone plates in the text have been prepared.

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[1] The Report has been published since the discussion of this Paper.—Sec. Inst. C.E.

Discussion.

Sir CLEMENT HINDLEY remarked that, as Chairman of the Steel Structures Research Committee, he was very glad to have the opportunity of making a few comments on the Paper. That Committee, as Members of The Institution were probably aware, was the result of a very remarkable piece of co-operation between industry and engineering science. The British Steelwork Association arranged with the Department of Scientific and Industrial Research for the investigation, and was partly responsible for financing the work. He had had a most interesting and valuable experience in being privileged to act as Chairman of that Committee, because it enabled a great deal to be learned about the way in which co-operation could be effected between manufacturers on the one side, professional engineers in the middle, and scientists on the other side. He felt sure that the members of the Steel Structures Research Committee, many of whom were present that evening, would be very grateful to the Author for having put forward such a lucid explanation of the Report of the Committee, which was now in the press. Sir Clement Hindley.

He wished to say a word in explanation of the circumstances in which the Paper had been presented. The Author had been invited by The Institution to present the Paper at a time when it was hoped that the Final Report of the Committee would have been published, and it was arranged specially so that the work of the Committee could be brought prominently before the Members, in the hope that they would have the Report in their hands at the time. The delay in the publication of the Report had, however, put the Author of the Paper in a slightly awkward position, in that he had undertaken a description not only of his own research work but of the research work of others who had helped the Committee before those others had had an opportunity of publishing and describing their own work.

In particular, Sir Clement wished to mention the most important original work done by Professor Batho for the Committee. In fact, Professor Batho and the Author had carried out the major portion of the practical investigations. There were others whose services Sir Clement wished to take the opportunity of acknowledging, because he felt himself particularly fortunate in having been connected with them and having seen something of their work.

Sir Clement Hindley.

He would like to make special reference to Mr. Ralph Freeman, who acted as Chairman of the Panel concerned with the research work of the Committee; Mr. Freeman had put in an enormous amount of work. Other names which should be mentioned were those of Mr. B. L. Hurst, Chairman of the Practice Panel, Professor Sir Harold Carpenter, Chairman of the Welding Panel, Mr. E. W. Butler, Dr. Oscar Faber and Professor A. J. S. Pippard, as well as that of Dr. H. J. Gough, of the National Physical Laboratory, who carried out the investigations connected with welding. Sir Clement also wished, on behalf of the Committee, to pay a very special tribute to the Secretary of the Committee, Mr. A. Zaiman, who first worked as Secretary when he was an officer of the Building Research Station, and later when he was a member of the staff of the Ministry of Health; the work was of a highly technical character, and the secretarial duties were onerous.

Sir Clement considered that the investigation was of outstanding importance to the engineering profession, and he did not think that in the last 40 or 50 years there had been one of more importance, or one which might affect more definitely the work of engineers engaged on design; if the more rational method of design could be brought into practice, the result would be of the very greatest importance and value. The work which the Committee had been able to publish in its First Report, although based on incomplete scientific data, had already been of immense benefit to designers and manufacturers, and he felt certain that the work as a whole would constitute a very great step forward.

Professor Batho.

Professor CYRIL BATHO remarked that the research undertaken for the Committee in the Civil Engineering Department of the University of Birmingham, and referred to by the Author, had been carried on during the whole lifetime of the Committee. Indeed, it might be said to have started before that, since in the First Report an account of an investigation on riveted and bolted splices, subsidized by an earlier grant from the Department of Scientific and Industrial Research, was included. That investigation was of more importance in connection with bridges than with buildings. The main work on buildings was the investigation of the behaviour of beam-and-stanchion connections, including encased connections, the development of the new method for the design of beams, and the experiments on bolted connections.

When he commenced the work on bolted joints he was astonished to find the small amount of information available on the subject, and he hoped that others besides structural engineers would find the papers in the Second and Final Reports of some interest. Besides the determination of the slipping loads and safe torques,

other subjects, such as the relation between torque and tension, the partition of the torque between the shank of the bolt and the washer, and the friction coefficients of the thread and the washer, were dealt with. He also hoped that the method of erecting steel frameworks with black bolts (preferably of high-tensile steel), tightened up to a given torque, would be given a fair trial, since it appeared to possess many advantages. The noise of riveting was avoided, the clearance around the bolts facilitated erection, and the beams could be designed under the new rules as economically as with riveted joints. The departure from ordinary methods of erection was not so great as might be assumed. The erection could be carried out with an ordinary spanner in the first instance, the bolts being afterwards tightened up to the required torque with a torque-control spanner, instead of being simply inspected for tightness as at present. Although he had found little information on bolted joints, the situation was worse with regard to beam-and-stanchion connections, since most of the information available was incorrect and misleading. It was only after long and complicated investigations that it had been found possible to elucidate the behaviour of such connections.

Professor Batho.

Regarding the new method of design, the statement had sometimes been made that by economizing in the beams greater moments were thrown upon the stanchions, and that, in consequence, the stanchions had to be made heavier. That statement had no foundation in fact. The substitution of a lighter beam in most cases left the moment transmitted to the stanchion practically unchanged, and sometimes even diminished it.

Another important point was that, although the nominal working-stress in the new method of design for beams was 9 tons per square inch, the actual maximum fibre-stress in the beam could seldom exceed 8 tons per square inch, and was often lower, as would be realized from *Fig. 28* in the Paper. If the beam-line corresponding to a stress of 9 tons per square inch were drawn it would always intersect the lower-limit curve at a higher point than the design curve; thus the restraining moment at the end of the beam was always greater than was allowed for in design. A satisfactory feature was that with normal angle-connections a considerable overload was possible before a stress of 9 tons per square inch, which was not in itself dangerous, was reached. That factor had an important bearing upon the questions of side-sway and wind-loads, and it was probable that the proposed method of design was unduly conservative on those points as far as the beams were concerned.

A great deal of important information had been obtained from the tests on existing buildings, but those tests had been of little use in developing the method of beam-design, although very useful in

Professor Batho.

showing that what had been found in the laboratory was also found in the field. The chief reason why they could not be used in the production of the design-method was that the ordinary working loads could not be exceeded, and thus no measure of the factor of safety could be obtained. That might have been done on the experimental frame at the Building Research Station, but work on that was abandoned when the Author left the Station.

The characteristics of the cleats and the effect of overload were determined from the laboratory investigations on connections, but it was also considered desirable to carry out laboratory experiments on a full-scale frame. Since the Author had not referred to those experiments, it might be of interest to mention briefly one or two points brought out by them. A full account would be found in the Final Report of the Committee.

The frame was made up from 12-inch by 5-inch by 30-lb. R.S.J.'s, and consisted of two stanchions 22 feet 10 inches long and placed at 15 feet centre to centre, connected by three beams. Tests were carried out with the frame in a horizontal position supported on rollers in the 300-ton testing machine in the laboratory, loads being applied at the third-points of the middle beam. A large number of experiments was carried out with $\frac{3}{8}$-inch, $\frac{1}{2}$-inch and $\frac{3}{4}$-inch angle-cleat connections having high-tensile bolts tightened up to given torques. In all cases the moment—angle curves deduced from the observations were in good agreement with those given by the tests on standard specimens, whilst the stresses at mid-span of the loaded beam were in remarkably close agreement with those predicted by the method of analysis given in the paper in the Final Report which dealt with the design of beams, using the average moment—angle curves obtained from tests on standard specimens, and not the lower-limit curves.

Professor Batho then showed a number of lantern-slides giving some of the results of the tests. *Fig. 36**, for $\frac{3}{8}$-inch cleats, was reproduced from one of them.

The maximum flexural stress in the loaded beam reached 8 tons per square inch at loads ranging, with the different connections, from 35 to 60 per cent. higher than those given by the present method of design, and did not reach 8 tons per square inch under the loads given by the proposed new method. That method gave safe loads ranging from 27 to 50 per cent. higher than those given by the present method.

The effect of overload might be seen from the following example.

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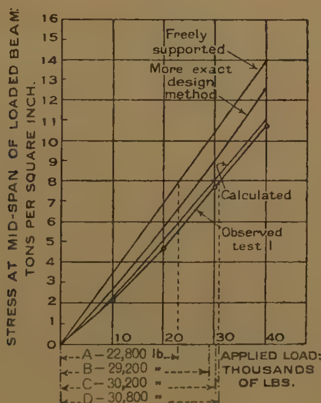
The stress at the working load given by the new design-method in a particular experiment was 7·5 tons per square inch. The load was increased by 37 per cent., removed and reapplied. The stress in the beam at the working load increased to 7·9 tons per square inch, but that increase was of no importance, since the same overload could again be applied to the beam without increasing the stress beyond the original value at that load. The load-factor of the beam was not diminished by any overload less than that factor multiplied by the working load. Much valuable information was obtained on the frame regarding the local stresses in the stanchions, the slopes of the stanchions being measured along their whole length. Professor Batho.

Fig. 36.



A word might be said about the production of the lower-limit curves. The most important factors causing variation in behaviour of similar connections were abnormalities in the shape of the cleat and differences in the tension of the bolts or rivets. It had been discovered that by making the cleats artificially acute or obtuse it was possible to find the range of the variations due to those abnormalities, and when that was found it was possible to explain almost all the anomalies of behaviour (as they had been considered to be), which had occurred previously in the experiments. Unfortunately, those experiments took place near the end of the investigation. After that, it was found that if in a particular set of experiments cleats all cut from the same piece were used, and the bolts were uniformly tightened, there was very little variation in the behaviour of similar connections.

Mr. M. B. Buxton said that the value of the Paper lay in the fact that it dealt with a subject about which there was very little information. A great many books had been written about bridges, but Mr. Buxton.

Mr. Buxton. very few about steel-frame design, in spite of the fact that the constructional engineering industry was now of greater importance nationally than the building of bridges, for it employed more men and used more steel.

He would like to refer to the section of the Paper headed "Application of the Method of Design." Design was very important, as all engineers were aware; but although the Author said that present methods left much to be desired, it was not possible to point to any single failure, and it was necessary to be very careful, from a national point of view, that design did not become too complicated and too expensive. An instance of that was the Stock Exchange building. That building had been put up more than 50 years ago, and he had it on the authority of the Chairman of the company which put it up that it was largely supported on wrought iron, there being a very big roof supported on wrought-iron lattice girders. At the present day, if such a design were submitted for approval it would be rejected by the authorities on the grounds that it would not stand, but the Stock Exchange had withstood the weather for 50 years.

The Author would be the first to desire that his Paper should be of great value to constructional engineers and consultants, but the method of design it suggested was long, and might be too difficult for the ordinary engineer employed in the industry. Mr. Buxton would particularly like to ask the Author, therefore, whether it would not be possible to have a design which was a little less complicated. If that could be arranged it would be of real service to the industry, because then it would be possible to have greater economy.

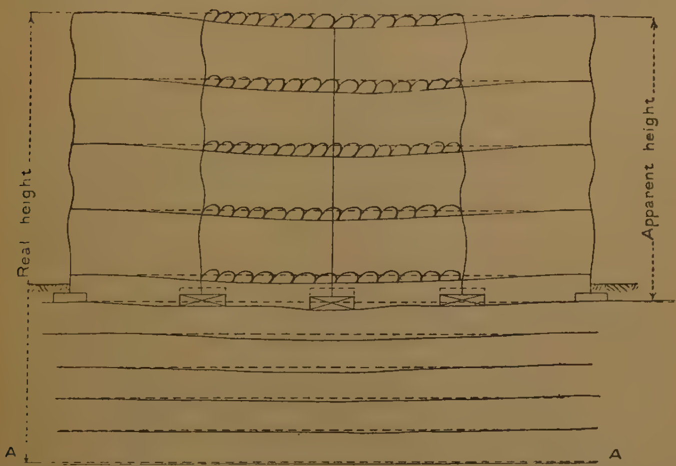
The Author drew attention on p. 167 to the great effect on the steel frame of clothing it with walls and concrete casing, an effect which was familiar in practice. If the more rational design suggested by the Committee were to be employed, Mr. Buxton would like to ask the Author whether it would not be reasonable to have higher stresses, of perhaps 9 or even 10 tons per square inch, and to have reduced loads, and whether it would not be reasonable to take account of the effect of the casing in strengthening the steel.

Mr. Manning. Mr. G. P. Manning said that the real problem concerned with the steel-framed structure was to know what was the ultimate strength of the complete structure under all conditions of loading. What happened under working loads was only a matter of minor importance. As for what was so far known about the ultimate strength of the whole structure, several speakers had said that a very large number of structures had been designed on certain assumptions and they continued to support their loads in a satisfactory manner. It might be asked what more an engineer wanted

to know, and the answer would be: only whether it was possible to build cheaper structures which would still continue to support their loads in an equally satisfactory manner. In saying "cheaper," he meant in total cost, including the overhead and drawing-office costs. Mr. Manning.

Why was not it easily possible to arrive at the ultimate strength of the complete structure? It was easy to arrive at the ultimate strength of any isolated member under artificial end-conditions by merely loading it to destruction in a testing machine. The main difference between an isolated member in a testing machine and a similar member, or the same member, in a frame was that the end-conditions, comprising the thrusts, moments, and shears imposed

Fig. 37.



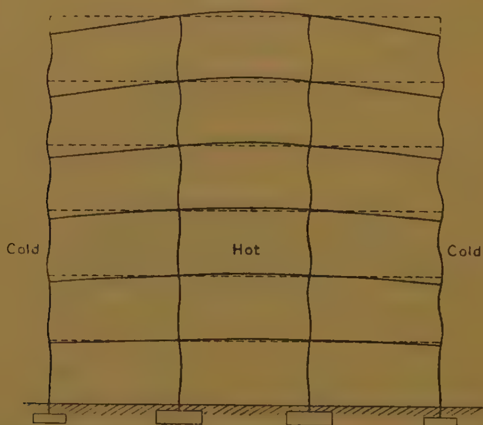
on that member by the other members at the panel-points, were not known. Those moments, thrusts, and shears were caused by relative movement of the members which framed into one another at each panel-point. If there were no movement of any member at any panel-point, then each member would be purely fixed-ended, and any loading applied to any one member would not affect the stresses in any other member whatever.

In general, there were three movements of each panel-point, namely, (1) rotation, (2) vertical movement, and (3) horizontal movement. Of these, the first was discussed in the Paper, the second was not discussed, and the third, apart from one or two references to structural side-sway, was also not discussed.

One cause of relative vertical movement was shown in *Fig. 37.* Quite apart from local settlement of the foundations, which always took place in any structure, there was always a more or less elastic

Mr. Manning. compression of the ground under the footings. It was impossible properly to consider a structure without taking into account the soil-mechanics of the site on which it stood. If the building were actually supported on piles, the elastic vertical compression had to be considered down to a level somewhere near the point of the piles, but even if it stood on ordinary ground there were columns of compressed ground under the actual footings, and the real height of the structure was as shown in *Fig. 37*. Those vertical movements were intensified in buildings with only two or three floors, because there was so much more opportunity for that type of partial movement, and they were most intense where there was only one suspended floor. They became more and more important as the ultimate load of the structure was approached.

Fig. 38.



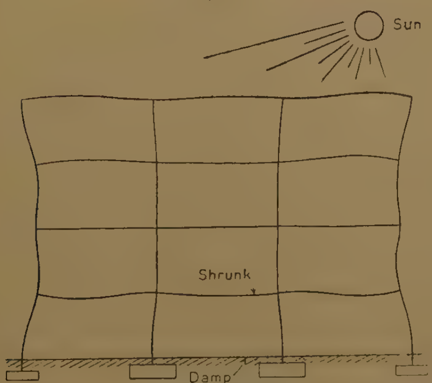
Another cause of vertical movement of the panel-points was shown in *Fig. 38*. The change in height of the columns due to unequal temperature could be measured by micrometer readings of the floor-heights, and was quite sufficient to put machinery on the upper floors out of level and to put shafting out of alignment.

Two causes of horizontal movement of the panel-points were shown in *Fig. 39*. If the structure had reinforced-concrete floors, all the suspended floors would shrink heavily. That shrinkage was resisted by the steel framing and partially relieved by cracking of the floors; nevertheless, if a long building were built in sections the joints certainly did open. In a complete reinforced-concrete structure the bending moments in the columns due to shrinkage were often many times the bending moments in the columns due to frame-action, and in that connection it might be pointed out that, although

the regulations asked for the structural frame-action to be taken into Mr. Manning. account, they seemed to forget the other moments, which might be two or three times as big. That horizontal movement was often important. The upper part of the diagram showed the effect of a hot sun, or conversely of a fall of snow on the roof. The only case of structural distress which he had ever seen in a column was due to that cause. It was a building designed for future extension, and it had a very heavy flat roof supported on abnormally stiff columns.

There were other causes of movement to which time would not permit him to refer. In his opinion, the failure to discuss those vertical and horizontal movements detracted very largely indeed from the value of the Paper. The importance of them varied from structure to structure, but, as only one of the three was discussed

Fig. 39.



in the Paper, it might be said that only one-third of the possible field of investigation was represented.

There was, however, another and much more serious objection, namely, that with the exception of the stanchion-connections and, he thought, one other reference, the experiments were limited to stresses which were well within the elastic limit. The range of stress discussed in general, therefore, was less than half the total range. It was necessary to know the ultimate strains of the structure, and the range of the strains was some fraction less than one-half of those. So far as the possible field of investigation was concerned, therefore, stresses below the elastic limit were discussed, but stresses above the elastic limit were not discussed. He had already shown that only about one-third of the field was covered so far as joint-movement was concerned, and, taking the two together, that meant that the investigations covered less than one-sixth of the total field.

The experiments described in the Paper were most interesting

Mr. Manning. as experiments, and he was most grateful to the Author for publishing them. He realized that the subject was very extensive, but he did not think that the range of the experiments was anything like sufficient to justify any conclusions concerning the ultimate strength of the structure ; he would say that they represented possibly 10 per cent. of the total field. Speaking as a designer, he very much resented having his hands tied by any regulations, and particularly when he thought that they were based on inadequate investigation.

The Author had attempted to make use of and modify the existing theory of the elastic structure ; Mr. Manning could not agree with him in that, being of opinion that what was required was not a modification of the existing theory of elastic structures but a completely new theory, based on the conditions in the total structure at its ultimate load.

He thought that the Author's outlook on columns was wrong both in principle and in detail. In principle the Author's views would apply only to a column loaded in a testing machine or to columns in a structure where they had a factor of safety of 2, or less, and the beams, cleats, and foundations had a factor of safety of 4, or more, so that the columns reached their ultimate stress before the beams, cleats, and foundations reached the yield-point.

In conclusion, he would like to ask four questions. Had the Author any evidence which would show that his method of design would give a true picture of the ultimate strength of the whole structure ? Flat-slab buildings were generally more complicated than steel building-frames, yet the Americans had succeeded in reducing their design to a set of simple rules. Could not similar rules be evolved ? Could the Author give some relative costs of a structure designed as at present and one designed on his assumptions ? Lastly, if the Author was prepared to ignore high working-stresses in cleats, which he said could be treated as a unit, why should he not ignore high working-stresses in columns also ?

Mr. Grunspan. Mr. A. S. Grunspan associated himself with the thanks which had been offered to the Author for a very interesting and important Paper. The Author mentioned that the stresses due to the eccentricities were in some respects from twenty to thirty times the calculated stresses. It was fairly obvious that those stresses were of such a nature that the structure must adjust itself to them, just as, for example, rivets adjusted themselves to high stresses when out of alignment or members adjusted themselves to stresses due to pre-heating or pre-stressing of the material.

He would like to ask whether the method suggested by the Author was going to lead to any more economical results. If the beams were increased, as was done by treating them as freely-

supported members, the bending moments in the columns were decreased as compared with the method of design suggested. That was a condition which had to be contended with at the present moment, and yet it was known that the columns were strong enough to withstand safely the stresses imposed upon them. In the case of buildings, it might be said that the columns did not carry their designed loads, because the designed floor-loading was but seldom reached, whether in the case of warehouses or of office buildings; in the case of structures such as water-towers, however, the loads on the columns were those calculated. In the design of a large number of structures of that description, he had reduced the eccentric moments to what he thought were permissible limits consistent with economy, and over a period of years the results had been satisfactory. Mr. Grunspan.

He agreed with the view that if a method of design so complicated as that suggested were to be adopted, the position would be almost hopeless. At the present time it was necessary to work at great pressure and to get the work done quickly, and what was wanted was a practical method based on scientific principles.

Mr. J. R. PINKERTON said that, his interest in steel-work being economic as well as academic, he had studied the Paper to see what assistance he could derive from it. On the theoretical questions he did not propose to linger, and, since the statements made were backed up by the results of exhaustive experiments, in the absence of information to the contrary they would have to be accepted. The economic aspect, however, seemed to be a different story, and one conclusion to which he had come was that the rational design of steel building-frames must first be preceded by rational planning. That was going to be rather a difficult problem, because the planning was usually done by architects and engineers with the object of selling the building and not of selling the steel. An examination of the results given in the Paper would reveal that a stanchion which, when designed under the existing Code of Practice, or B.S.S.449 regulations, would weigh approximately 33 cwt. would, according to the example shown in the Paper, weigh 47½ cwt. That meant a saving of 14½ cwt., or 30 per cent., or, to put it more accurately, the rational design involved an increase in weight of stanchions of 45 per cent. Against that there was a saving in beam-design by the rational method of 2½ cwt., which would probably be enough to pay the designer's wages for finding the fixed-end moments for beam No. 102 of the example quoted in the Paper. The additional weight of cleats and bolts involved would also have to be taken into account. While, therefore, it was impossible to have anything but praise for the method of arriving at the results, it was possible, speaking from the economic standpoint, to wonder whether the end justified the means. With regard to the calculations, he would like to know why Mr. Pinkerton.

Mr. Pinkerton. a stress of 9 tons per square inch had been adopted for beam No. 21 and one of 8 tons per square inch for beams Nos. 92 and 102.

He did not wish to argue that, because buildings as designed in the present and past stood satisfactorily, the method of designing them was correct, but the fact that they did stand seemed to justify the assertion that they were strong enough, which according to the rational method of design was not the case. He would prefer to adopt the rational method of design, but only provided that it was rational in all respects, and he would suggest that, along with the more exact methods now proposed, a set of floor-loadings should be adopted which were more in accordance with the actual facts, so that the additional labour and expense of the calculations might have more justification. Steel-frame buildings had stood satisfactorily, and it was not always because of the factor of safety; it was, therefore, something to do with the loading. A figure of 50 lbs. per square foot for offices and a figure of 40 lbs. per square foot for flats were much too high. He had obtained the figure for the offices where he himself worked, and which had the average contents of City offices, and it came out at exactly 6 lbs. per square foot. Rationalization should therefore be applied in that respect, and, if investigation into design were to be so carefully carried out, he thought it should be continued so that other and equally vital points would be taken into consideration before any rational method of design such as that now proposed was adopted.

For the sake of the industry of steel-frame buildings, he suggested that the proceedings that evening should be kept as dark as possible. If they came, for example, to the knowledge of the London County Council and District Surveyors, the life of the average steelwork designer would not be worth living.

Mr. Grisenthwaite. ⁂ Mr. T. C. GRISENTHWAITE observed that the Paper, which was a result of extensive experimental research and theoretical analysis, presented a more exact method for the design of building frames; the Author was to be congratulated on his work on a most difficult problem, and on the thoroughness with which he had interpreted the results and set them out in a form which could be applied to actual design. On p. 136 the Author made it quite clear that the present method of design, whilst unsatisfactory, did produce safe structures, and stated that the problem was to find a more rational method of design based on experimental evidence, with the object of making further economies in material.

It was obvious that, so far as the beams were concerned, the new method did result in considerable economies. On the other hand,

⁂ This and the succeeding contributions were submitted in writing.—SEC. INST. C.E.

from the examples given at the end of the Paper, it was apparent that the sections of the stanchions would have to be increased. The reason for that was not far to seek if reference were made to the calculations given for stanchion No. 10 in Table XIV. It would be seen that, for the section between floors 5 and 6, the column had to sustain a vertical load of 34·1 tons, a bending moment on the XX axis of $169 + 73 = 242$ inch-tons, and a bending moment on the YY axis of $210{\cdot}3 + 104{\cdot}7 = 315$ inch-tons. Those moments were increased on the XX axis to 257 inch-tons, and reduced on the YY axis to 215 inch-tons, by certain factors given in col. 7 of the Table. The necessary section, with an end bending stress of 6·59 tons per square inch against a permissible of 7·3 tons per square inch, was given as a 10-inch by 6-inch by 40-lb. joist with two 10-inch by ½-inch plates. Under the present Code of Practice a vertical load of 34·1 tons with a bending moment on the XX axis of only 32 inch-tons and 6 inch-tons on the YY axis would require a stanchion of 8-inch by 6-inch by 35-lb. R.S.J., with a maximum fibre stress of 5·2 tons per square inch, against a permissible of 5·4 tons per square inch as obtained from Clause 43 of the Code. Mr. Grisenthwaite.

On p. 210 the Author suggested that local authorities should offer inducements to designers to adopt the more exact method available, and at the meeting he suggested that those might take the form of lower superimposed loads. That scarcely seemed a rational solution. A more logical method would be to maintain equality in floor loads under both methods of design but to increase the stresses to greater values if the rational and more exact method were to be adopted.

If a column designed under the Code of Practice were admittedly safe, where no account was taken of the restraining moments which undoubtedly existed, surely a considerable increase of stress should be permitted when those factors were taken into account. The margin between the safe stress and the yield-point existed to allow for uncertainties in loading and errors in the method of design and workmanship, such as (1) inexactitude in determination of stresses and neglect of secondary stresses, (2) errors in workmanship, and (3) uncertainties of the actual loading.

It was fairly clear from the calculations given for the stanchions that the inexactitude of design and the neglect of secondary stresses represented a considerable portion of the margin between the safe and the yield stresses. If those stresses were at present calculated by the rational method it would seem logical to increase further the stresses and yet to provide the same margin for errors of workmanship and uncertainty of the actual loading. The Author seemed to have had such an idea under consideration when he suggested, on pp. 197 and 199, that under certain conditions the load-factor should be less than 2, such a figure as 1·25 being quoted. Another point which

Mr. Grisenthwaite. scarcely seemed logical was the adoption of a stress of 9 tons per square inch for beam No. 21, which was restrained by its connection to the flanges of the column, whereas the stress was taken as 8 tons per square inch for beams Nos. 92 and 102, where there was no restraint owing to the connection being made to the web of the column.

It might be doubtful whether engineers would be inclined to adopt the rational method outlined by the Author if, after the additional labour entailed in the design, heavier columns would result. It was true that a considerable economy could be made in the beams, but whether that saving would outweigh the increase in stanchions was not apparent without designing a considerable number of buildings of different types and comparing the weights. It would be unfortunate, however, if, after the large amount of valuable experiment and research which had been done in connection with a more rational design, engineers were reluctant to adopt it, which might be the case if any doubt existed as to whether the resultant economies were commensurate with the additional labour involved.

Mr. Powell. Mr. E. G. S. Powell congratulated the Author on the clear case he had made out for a rational basis of design for steel building-frames. All engineers would agree that, in the light of the experimental work done in connection with the Paper, the present accepted method of design was grossly wasteful in beams, and that very often columns as designed were liable to be so overstressed, if the structure were loaded unfavourably, as to leave a very small factor of safety. Mr. Powell considered that every engineer should do all within his power to bring about the reforms necessary for the adoption of a rational basis of design. When it was realized that under present systems allowed by regulations it was possible to design a building with a very low factor of safety in the columns, and that the failure of a column might mean the collapse of the whole building, it would be seen how urgent was the need for reform. The fact that very few structures had failed did not reveal the defects in the method, as structures were very rarely loaded to the worst condition of their design-load.

Although the calculations entailed more work in the proposed method than had been usual in the past, it gave opportunity for economy in beams with a greater general factor of safety, and would thus allow floor-loads more near to actual loads to be safely used in design, whereas under the present system reduced floor-loads in design would almost certainly reduce the size of columns to the danger-point. Regulations governing the design of building-frames should be such that adequate strength was ensured, but they should allow the designer freedom to treat the structure accurately as a frame, and to effect any economies he could by his more accurate analysis.

Mr. Spencer. Mr. A. S. Spencer thought that the Author had dealt with a very

complicated subject in an exceptionally lucid manner, and he Mr. Spencer. was sure that all those connected with the design and construction of steel-framed buildings would be grateful for the valuable information supplied.

The tests and investigations were based upon structures resisting vertical loads only, and the secondary stresses found were due to the semi-rigid method of connecting the beams to the stanchions. Such connections could be avoided when only vertical loads were being considered, and thus the secondary stresses would not exist if the assumption were made that the clothing itself did not provide a degree of fixity. Structures of the type under investigation would have to resist horizontal wind-forces, and that resistance could be achieved by semi-rigid or rigid connections, or alternatively by suitable vertical bracing in the side and end walls, and in addition, where necessary, bracing across the interior of the building.

It was stated on p. 204 that the Committee finally recommended that the method of design "should apply 'to the steel framework for building structures formed with horizontal girders and vertical columns and provides only for the stresses caused by vertical forces. Any structure for which the method of design is used must be so constructed as to resist horizontal forces due to wind or other causes, without significant horizontal sway, by means of its floor slabs in association with vertical walls or braced vertical frames.'" In the early days of structural design the efficient resistance of horizontal forces had been neglected. In buildings of the type under investigation the usual practice had been to design the joint between the beam and the stanchion in such a way as to resist the vertical shear force of the floor-load, but without any significant rigidity such as would have set up secondary bending stresses in the stanchions. Practice had proved that those buildings were stable, but it would have to be admitted that the superimposed floor-loads for which the buildings were designed were excessive. Design-regulations which now existed allowed for lighter superimposed floor-loads but called for semi-rigid connections; there was no doubt that current practice was not a rational estimation of cause and effect, and that a rational change was necessary. The Author had proved in his Paper what was generally accepted by thoughtful designers, and he would have done great service if that necessary change were made at an early date.

If the fixing-effect of the covering materials could be ignored buildings could probably be designed in such a way that the vertical loads would not set up any serious bending stresses in the stanchions, and the horizontal forces could be resisted by suitable side, end, and interior bracing; as, however, there would probably not be any

Mr. Spencer. economy in that method as compared with the adoption of portal-action or a combination of portal-action and bracings, the latter method was perhaps the one that should be adopted. The latter method would probably be the most economical; it was the one recommended by the Committee, and it would appear that one type of structure could be adopted subject to the cause and effect of vertical and horizontal forces, including that of the covering material, or alternatively, of the same forces suited to the condition where the interior steelwork was left naked. The type of structure would be one which would have theoretical pin-joints in the stanchions situated at some vertical position between the two floors, and theoretical pin-joints existing in the beams somewhere adjacent to the stanchions. If that condition could be visualized then probably it would be possible to determine the position of those pin-joints, firstly for the steelwork clothed, and secondly for the interior steel-work unclothed. Those two systems would necessitate the adoption of suitable connections.

It was to be hoped that eventually it would be possible to formulate an economic and reasonably simple method of design, so that all those who were concerned in the design and construction of such buildings could be brought to agreement. Great Britain had been very slow to adopt a rational method of design such as was recommended, as most other countries, particularly Germany, now used rational methods of design.

The Author. The AUTHOR, in reply, apologized for having presented a somewhat lengthy Paper, and said that it had been impossible to compress the account of the Committee's work into a smaller compass. He was very glad that Professor Batho had been able to expand the information given on beams and connections. It should be remembered that a team had been working on the subject for about 6 years, and that, in spite of the length of the Paper, it had not been possible to cover all points.

Several speakers had asked for a more simplified method of design. By far the most difficult part of the task had been the reduction of the method to workable proportions without losing the rational basis; the Committee felt that it had gone quite far enough, and that further simplification would destroy that basis. He was tempted to remember that part of his duty was the training of engineers, and to suggest that there were men who were quite capable of applying what might appear to be a more complicated method. In actual fact, the method was not very seriously different from that at present used, although, naturally, structural designers, like everyone else, hated to change their habits. He thought that something might be done in future by producing sets of curves giving information in a form familiar to-day to designers of rein-

forced-concrete structures. Some of the labour of calculation might be relieved in that way, but he was quite certain that the method could not be further simplified without ruining its basis. The Author.

Reference had been made to the effect of casing and the possibility of using an all-round figure for steelwork stresses of 9 tons per square inch. The presence of casing, as the tests on existing buildings had showed, could have quite other results than reducing the bending stresses in the steelwork. The effect of casing was most complicated. Where there were heavy, rigid steelwork-connections the presence of clothing on the frame might reduce the bending stresses resulting from any applied load, but where there were fairly flexible steelwork-connections the effect of concrete placed round them might be, as the tests had shown, to increase the stresses by as much as three times.

With regard to the reactions at the ends of the members, attention had been by no means confined to those caused by rotations of the joints. The main purpose of most of the work had been the determination, in as complete a form as possible, of the end reactions under all arrangements of load on the complete structure. Relative movements of the foundations had to be considered when dealing with continuous structures; it was impossible to guard against such movements by drafting a clause in a Code of Practice, but the Committee's view was made plain, as would be seen from the Final Report, in the "Introduction to the Recommendations for Design," where it was stated that "The foundations of the structure must be so constructed that relative settlement of the various foundations under the action of dead and live load is practically avoided." In the office building which had been tested the levels of the footings of four stanchions were measured periodically, and up to the time that the full dead load and a certain amount of superimposed load were in position, no relative movement had been detected, although the stanchions all stood on very different types of ground. Movements due to normal temperature changes had no serious effect on the type of steel-framed structure to which the Recommendations were to apply, and, although no figures were available, the same could certainly be said of the effect of the shrinkage of the concrete casing.

He was in agreement with most of Mr. Manning's remarks, but Mr. Manning was wrong in supposing that his (the Author's) views on stanchions would apply only to a column loaded in a testing-machine. For what appeared to be the first time in any comprehensive design-method, recourse had not been made to the unsatisfactory basis of the pin-ended strut. Although sufficient space might not have been given to that side of the investigation, it should be clear, from *Fig. 27* and Tables VII and VIII, that each stanchion-length was considered as part of a continuous member

The Author. and that the stresses developed in it by loads on every other member in the structure, not only on the beams connected to it, were taken into account. On that basis each stanchion-length would be designed so that the yield-stress of the material was developed at the same time as that stress was developed in the beams, the "factor of safety" in stanchions and beams being the same and not, as suggested, in the ratio 1 : 2, while, as was made clear in the Paper, that "factor" for the connections was very much less. There was, also, ample evidence from the tests carried out on existing buildings that the method of design gave a true picture of the strength of the structure, based on yield-stress, although not, perhaps, of its strength based on ultimate stress.

The question of ultimate strength was certainly of interest, but the reason why only half the field, as suggested, had been covered in the tests on buildings was the very simple one that, in the interests of the building owners, who had very generously allowed the tests, great care had to be taken not to pass the yield-stress. Ultimate strength and higher working-stresses leading to the region beyond the yield-point might prove of the greatest importance in steel-frame design, as they were, although probably not so recognized generally, in the case of reinforced-concrete structures. Although the Committee had gone some way in that direction, as had been seen, where connections were concerned, it had not ventured on to the main members. The way was now clear, however, since the behaviour of that type of structure was well understood in the elastic range; that was an essential preliminary to further study of the inelastic range. The question had been asked as to why high working stresses should be accepted in connections and not in stanchions. Whilst it had been shown that such stresses were no source of danger in connections, the same could not be said of stanchions. Some credit might be given to a steel stanchion-length for its powers of standing after the yield-stress had been reached at its end sections, but that was not the whole problem. As pointed out in the Paper, a stanchion could bend in "double curvature" or in "single curvature." When it bent in "double curvature" the maximum bending stress might, and usually did, occur at the end, and, if the yield-stress of the material were passed under that arrangement of the loading, there would not be catastrophic failure. On the other hand, when the arrangement of the load produced bending of the stanchion in "single curvature," the maximum stress occurred at the centre of the stanchion length, and, when the yield-stress of such a member was reached, failure was not very far distant.

Attention had been drawn to the fact that stanchion No. 10 as designed in the Paper was heavier than it would be if the existing Code of Practice were used. The example of Table XIV had been

chosen because it showed clearly all the points in the new calculation-sheet. It would be most regrettable if the impression were given that the new method led everywhere to heavier stanchions. That was not so. A short example based on Table XIV should make the position clear. He would assume that stanchion No. 10 (*Fig. 35*) was an external stanchion with three beams Nos. 21, 92, and 102 framing into it, beam No. 21 carrying the loads shown in *Fig. 35*, but beams Nos. 92 and 102 being equal in all respects, carrying the same dead load but no live load, such that the total load on the stanchion length was 152·39 tons. The stanchion-section of Table XIV, a 10-inch by 6-inch by 40-lb. I-section with two 10-inch by ½-inch plates, would be found adequate, the calculations being as given in Table XV (p. 230). On the other hand, under the existing Code of Practice such a section would not be strong enough, since even assuming a ratio of effective to actual length of 0·75, the permissible working stress (Clause 40) would be only 6·66 tons per square inch. The same conditions would be found in every other length and the proposed method would therefore produce a lighter stanchion. It would appear, however, that, taking the figures for floor-loads and permissible stresses given in the Paper, there would be a tendency on the whole for stanchions under the new method of design to become heavier. That tendency could be limited to some extent by rational planning. There was, at the same time, a strong case for urging local authorities to help, possibly by means of a reduction of superimposed load, or, as Mr. Grisenthwaite had suggested, of the load-factor, to bring even the heaviest stanchion as given by the proposed method more nearly to the weight of that designed to-day, on the evidence of the fact that the stanchion to-day gave no cause for anxiety. The Author.

Several speakers had drawn attention to the fact that, while 9 tons per square inch had been adopted for beam No. 21, beams Nos. 92 and 102 were designed to 8 tons per square inch. That was in accordance with the Committee's rule that where allowance was made for the restraining moments in accordance with Clause (2) and (3) (pp. 185, 186) 9 tons per square inch might be used, but where no allowance was made the existing 8 tons per square inch would have to be used. The reason, which appeared somewhat inadequate now, was the strong feeling that the proposed method should be applied as a whole. If an overall 9 tons per square inch had been allowed wherever beams were fitted with cleated connections there might have been a tendency for that stress to be adopted generally for beams without any account being taken of restraining moments and their effect on the stanchions. When the proposed method was adopted it would be as well to treat all beams designed under it in the same way and to use 9 tons per square inch throughout as the nominal permissible stress.

The Author.

TABLE XV.

Col. 1	2	[illegible]	4	5	6		7		8
	I-section 10 inches × 6 inches × 40 lbs. Two 10-inch × ½-inch plates. ← 9 feet 3 inches. →	21 $R^D = 4{\cdot}4$ $R^L = 1{\cdot}9$ $M^D_F = 145$ $M^L_F = 62{\cdot}5$ $R^D = R^D$ $R_L = 0$ $R_L = 0$		$p = \frac{152{\cdot}39}{21{\cdot}77} = 7{\cdot}0$ $\frac{l}{r} = \frac{111}{2{\cdot}19} = 50$	XX	D, $145 + 4{\cdot}4 \times 5{\cdot}5 = 169$ L, $62{\cdot}5 + 1{\cdot}9 \times 5{\cdot}5 = 73$	$f = \frac{1}{87{\cdot}3} \times \frac{1}{2}[0{\cdot}97 \times 169 + 1{\cdot}28 \times 73]$ $= \frac{164+93}{174{\cdot}6}$	$= 1{\cdot}47$	$\frac{12{\cdot}5}{2 \times 51{\cdot}9}$ $= 0{\cdot}12$
		$M^D_F = M^D_F$ 92 — 102 $M^L_F = 0$ $M^L_F = 0$			YY	D $= 0$ L $= 0$		$= 0$	$\frac{34{\cdot}4 + 2{\cdot}9}{2 \times 11{\cdot}4}$
5.			152·39	$f_A = 1{\cdot}6$				Total $= 1{\cdot}47$	$= 1{\cdot}63$

The following Paper was read and discussed at a meeting of the Birmingham and Distict Local Association of The Institution of Civil Engineers on 12 March, 1936.

Paper No. 5058.

"Some Developments in Railway-Carriage and Wagon Construction."

By Paul Lewis Henderson, Ph.D., B.E., Assoc. M. Inst. C.E., A.M.I.Mech.E.

TABLE OF CONTENTS.

Introduction.

The first railway coaches constructed in 1825 for the Stockton and Darlington Railway were very similar to road vehicles of that time. Development for many years was very slow due to the Government restricting the gross tonnage of all vehicles to 4 tons. This restriction was, however, repealed in 1842, and carriages gradually advanced in size and comfort until in 1874 bogie coaches

were introduced into this country from America by the Pullman Company.

Although the first steel coach was introduced about 1904 and a number have been built since, the bulk of the coaches constructed for use in Great Britain are of composite timber and steel construction. A typical British passenger coach is one having a steel underframe and bogies, but having the body-framing and inside finishing of timber and either timber or steel outside panels. The steel underframes and bogies have in the past been constructed of rolled sections with riveted joints, but a change over to welded construction is now being seen in common with railways in other parts of the world.

It is proposed in this Paper to describe the design and construction of the first coach having welded underframe and bogies to run on an English railway. This coach was built by the London, Midland and Scottish Railway Company, and was put into traffic in May, 1934. It has an entirely new type of body-framing which departs in principle from what has been standard in England for nearly a century. In the course of developing this body-framing, many timber tests were carried out and some of these will be described.

Many and varied have been the developments in wagon-construction, but probably one of the most important from an operating point of view is the introduction of the welded wagon, with its consequent reduction in weight. In the latter part of this Paper details will be given of the first welded 12-ton open merchandise wagon built by the L.M.S.R.

Experimental Investigation of the Body-Framing of Passenger Coaches Employing Timber Construction.

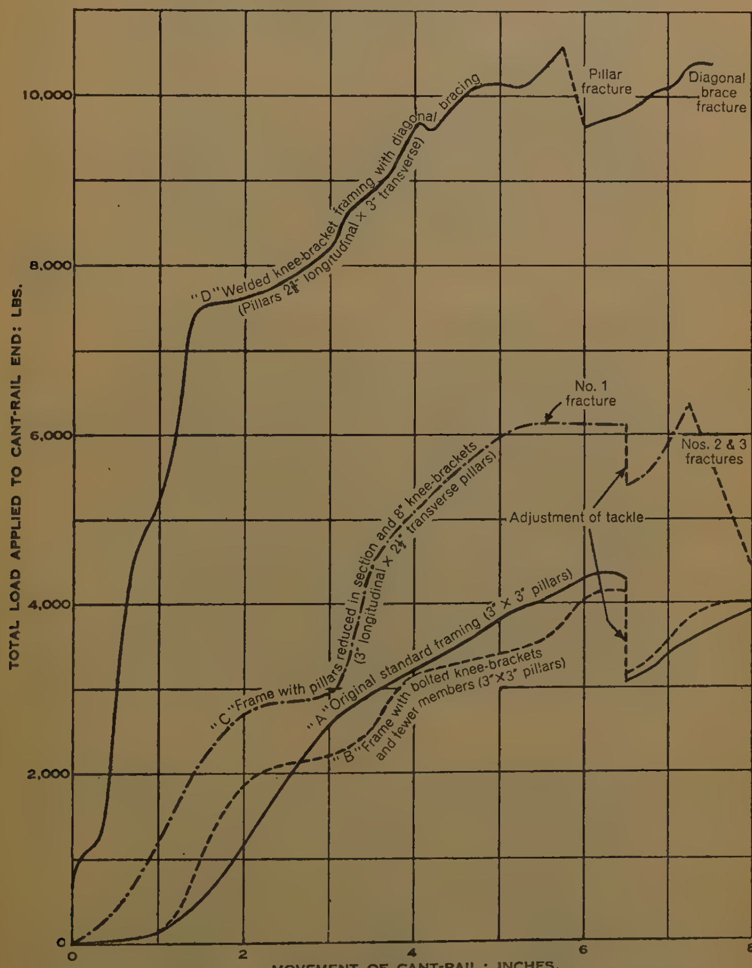
The first test carried out by the Author consisted of applying a horizontal force to the cant-rail of a 13-foot section of standard teak body side-framing, which was mounted on a length of bottom-side secured to the floor. The framing was made up in the same manner as that for a 57-foot corridor third brake. The force was applied to the cant-rail by means of a hydraulic lifting-jack fitted with a pressure-gauge, and the load was recorded for each inch movement of the cant-rail. The result is shown as curve A in *Fig. 1*, and the failure of the framing in *Fig. 2* (facing p. 234).

On examining the framing after the test it was seen that the coach-screws in the body-knees had given way, and the pillar-tenons into the timber cant-rail and bottom-side had failed in a number of places, with the result that the framing was pushed over like a pack of cards. The test revealed very clearly the inherent weakness of the mortise-and-tenon joint method of securing the

pillars, as the latter were practically undamaged and their maximum strength had obviously not been developed.

Next a similar section of framing was made up with a number

Fig. 1.



LONGITUDINAL SHEAR-TESTS OF CARRIAGE SIDE FRAMING. (SECTIONS TESTED 13 FEET LONG.)

of the cross timber battens omitted, and bolts were used in the body-knees in place of coach-screws. The result of this test is shown as curve B on *Fig. 1*, and it will be seen that this section of

framing is equally as strong as the original framing. Based on the result of this test, about one hundred and six members were eliminated from the standard framing of the 57-foot corridor-third-brake coaches, with a considerable saving in cost. It might be noted that the elimination of the timber members from the above coaches did not affect the column strength of the framing or the strength in a direction transverse to the centre-line of the coach. The slight loss in longitudinal shear of the framing was more than made up by the increased strength obtained by replacing coach screws by bolts as previously mentioned.

A section of framing was then tested having the following alterations from the standard L.M.S. Rly. framing :—

(1) Body knee-brackets increased in size from 6 inches by 6 inches by $\frac{3}{16}$-inch steel to 8 inches by 8 inches by $\frac{1}{4}$-inch steel, and two more used.

(2) Bolts with washers used in place of coach-screws.

(3) All pillars reduced from 3 inches by 3 inches to 3 inches by $2\frac{1}{2}$ inches.

(4) Cross battens reduced from $2\frac{1}{4}$ inches to $1\frac{1}{2}$ inch in thickness.

(5) Half-lapped joints were employed throughout, except for the top and bottom light-rails.

(6) Unnecessary members previously mentioned were eliminated.

The method of testing was similar to the first test. The result of this test is given by curve C in *Fig. 1*. It will be noticed that this frame is about 100 per cent. stronger in shear up to 2-inch deflection or movement of the cant-rail relative to the bottom-side, decreasing to about 40 per cent. stronger at 6-inch movement. The increased shear strength is due to the greater holding power of a larger knee-bracket with the addition of bolts and washers in place of coach-screws. The strength of coach body-framing depends largely on its resistance to longitudinal and transverse shear, which in turn are mainly governed by the efficiency of the method adopted for securing the pillars top and bottom.

It was early appreciated that a pillar with a mortise-and-tenon joint should be replaced by a plain-ended pillar fitting into a steel box-bracket. The first type of box-bracket which the Author tried out was made in two halves in between which the pillar was bolted, the box-bracket being then bolted to the bottom-side as shown in *Fig. 3*. Various other types of box-brackets were tried out, and *Fig. 4* shows two of these on the right-hand side. The knee-bracket fastening on the left-hand side of this figure is the standard method of securing the pillars, except that coach-screws have been used in

Fig. 2.

TEST ON BODY SIDE-FRAMING.

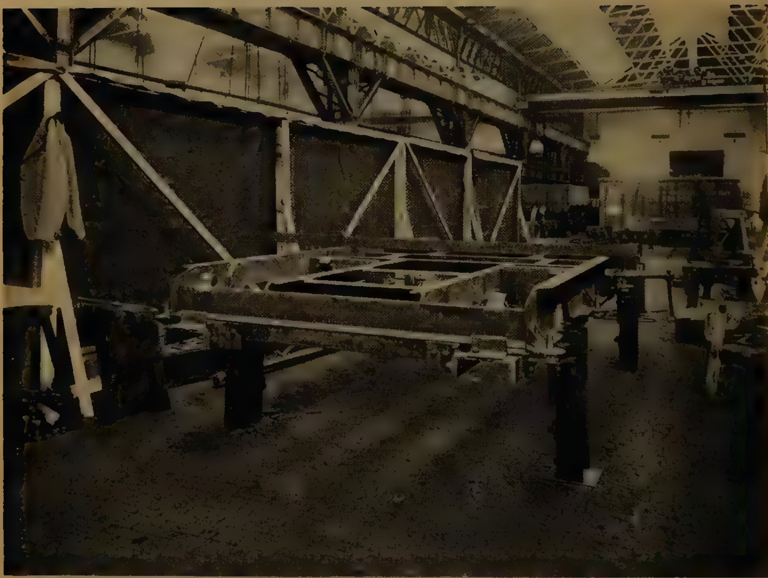
Fig. 3.

INITIAL DESIGN OF BOX-BRACKET.

Fig. 4.

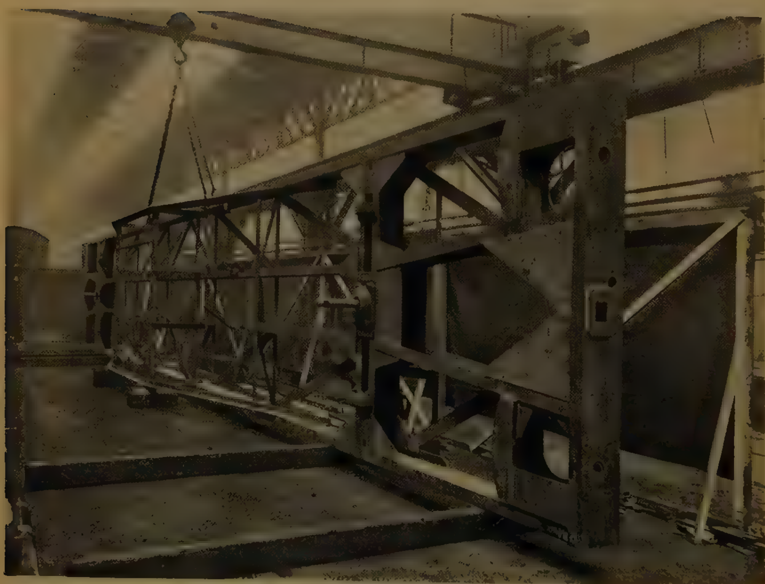
ALTERNATIVE DESIGNS OF BOX-BRACKETS.

Fig. 19.



CONSTRUCTION OF WELDED BOGIE-FRAME.

Fig. 20.



WELDED UNDERFRAME COMPLETE.

the past in place of bolts. The box-bracket in the centre of *Fig. 4* was finally adopted. After this a 13-foot section of framing, similar in design to that used for the coach which will be described later, was tested in a like manner to the first test of *Fig. 2*. In each of these tests of coach body-framing, the cant-rail has been increased in size so that the failure would not take place in that member while undergoing the test, but somewhere in the pillars. The pillars were $2\frac{3}{8}$ inches by 3 inches and were secured by $\frac{1}{8}$-inch welded steel box-brackets to a steel solebar, and by $\frac{3}{32}$-inch welded steel box-brackets to a steel-angle cant-rail. The diagonal bracing is of $1\frac{7}{8}$-inch by $\frac{1}{8}$-inch steel welded to the solebar and cant-rail. The framing was given a shear test by means of a hydraulic jack applied to the cant-rail while the solebar remained in position fixed to the floor. The deflection or movement of the cant-rail was measured by a plumb-bob hanging from it. The results of this test are given by curve D of *Fig. 1*. It will be seen that a 1-inch movement of the cant-rail requires about 5,000 lbs. as compared with about 150 lbs. for a similar section of the standard framing. It was found after the test that the welded box-brackets had not failed, and also that some of the teak pillars had failed through a full section above the box-bracket, and not by splitting at the end or by drawing the bolts.

Then a number of pillars were tested as cantilevers to ascertain the strength of various types of pillars and fastenings. The pillars in each case were fastened to a short length of bottom-side, the latter being secured to a vertical column with the pillar projecting out at right angles. A bucket was secured to the end of the pillar 5 feet 11 inches from the bottom-side. Readings of deflection of the pillar were taken 6 feet $7\frac{3}{4}$ inches from the bottom-side for increments of load, which were obtained by putting known weights into the bucket. Pillars were tested in longitudinal and transverse directions. By transverse direction is meant the loading of the pillar at the top in a direction across the centre line of the coach, whereas by longitudinal direction is meant that the loading is parallel with the centre line.

The following pillars were tested in a transverse direction:—

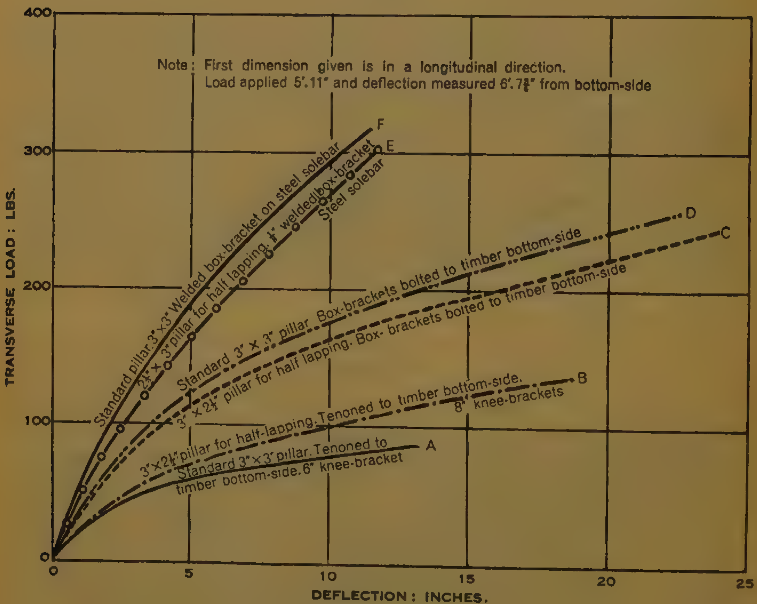
(A) Standard 3-inch by 3-inch teak pillar with an ordinary 6-inch knee-bracket secured by $\frac{3}{8}$-inch bolts in place of the usual coach-screws. The pillar failed by splitting along the grain from the tenon. The results of this and of the following tests are shown in *Fig. 5* (p. 236).

(B) To improve on the previous method of fastening, an 8-inch by 8-inch knee-bracket of $\frac{1}{4}$-inch instead of $\frac{3}{16}$-inch steel was tried out.

The pillar was reduced ½ inch in thickness, and cross rebates were made for half-lapping: washers were put under the heads of the bolts securing the knee-brackets to the pillars. The pillar failed in a similar fashion to (A), but was slightly stronger.

(C) Next the Author tried out the steel box-bracket fastening of *Fig. 3*, with 3-inch by 2½-inch pillars having cross rebates for half-lapping. The results were very surprising, as it was found that this method of fastening was about 70 per cent. stronger at 5-inch deflection, and took about three times the load to fracture it.

Fig. 5.



Coach-Pillar Tests.

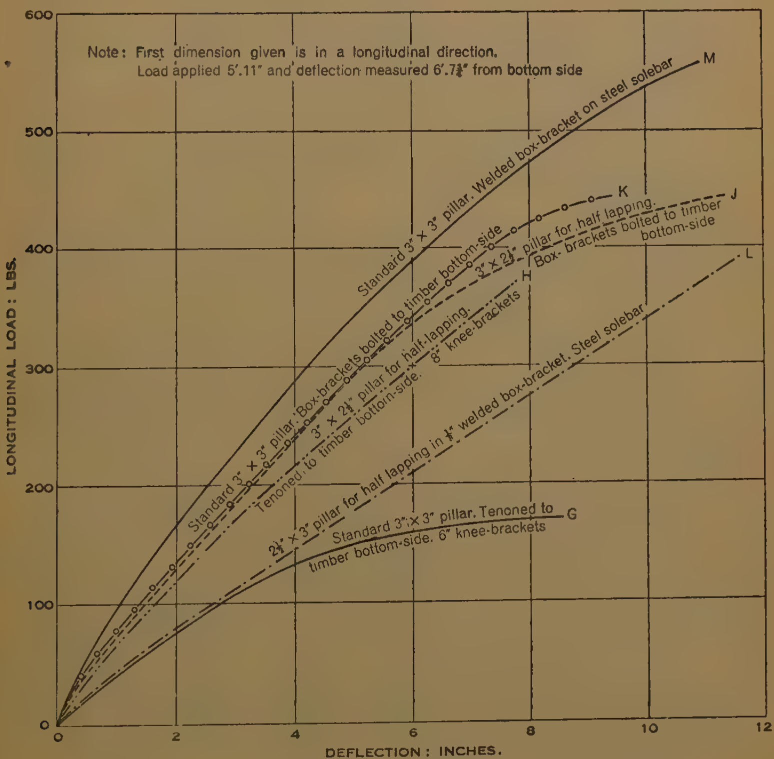
(D) As the 3-inch by 2½-inch pillars were used in the previous test, it was decided to repeat the test with 3-inch by 3-inch pillars in order to obtain a true comparison with the standard 3-inch by 3-inch pillar and the usual mortise-and-tenon fastening.

(E) In this case 2½-inch by 3-inch pillars with cross rebates for half-lapping and secured in the latest type of box-bracket welded to a piece of steel solebar were tested. The results in this case greatly exceeded any of those of the previous types of fastenings for strength, as will be seen on referring to *Fig. 5*. The 2½-inch by 3-inch pillar is considerably less in section than the standard 3-inch by 3-inch

pillar, and the former was also weakened by cross rebates for half-lapping as compared with mortise holes in the latter.

(F) 3-inch by 3-inch pillars of the standard type with mortise holes but secured in $\frac{1}{8}$-inch steel box-brackets welded to the steel solebar similar to (E) were tested. The great increase of strength due to the

Fig. 6.



COACH-PILLAR TESTS.

box-bracket fastening will be seen by comparing curve (F) with curve (A) of *Fig. 5*.

The following pillars were tested in a longitudinal direction, and the results are given in *Fig. 6* :—

(G) Standard 3-inch by 3-inch pillars with 6-inch knee-bracket. The pillar failed at the tenon.

(H) 3-inch by $2\frac{1}{2}$-inch pillars with cross rebates for half-lapping, and secured to the bottom-side with 8-inch knee-brackets. This

increased the strength of the pillar, but failure took place at the tenon.

(J) 3-inch by $2\frac{1}{2}$-inch pillars with cross rebates for half-lapping, and secured to the bottom-side in steel box-brackets of the type of *Fig. 3.* Failure took place by splitting from the bottom cross rebate.

(K) 3-inch by 3-inch pillars of the standard type, but secured in steel box-brackets similar to *Fig. 3.* Failure took place by the box-bracket bolts pulling into the jarrah bottom-side. The results of tests of pillars (J) and (K) show increased strength as compared with the standard pillar fastening.

(L) $2\frac{1}{2}$-inch by 3-inch pillars with cross rebates for half-lapping and in $\frac{1}{8}$-inch welded box-brackets similar to (E), and shown in the centre of *Fig. 4.* On referring to *Fig. 6* and curve (L), it will be seen that this pillar, although it is $\frac{1}{2}$ inch less than the standard pillar in the longitudinal direction in which it was tested, yet is equal to it in strength as a cantilever at 2-inch deflection, gradually increasing until at the point of failure the $2\frac{1}{2}$-inch by 3-inch pillar with the welded box-bracket had carried twice the load necessary to cause failure of the 3-inch by 3-inch standard pillar.

(M) 3-inch by 3-inch pillars of the standard type with mortise holes but secured in welded box-brackets similar to (L). The great increase in strength due to the box-bracket fastening will be seen by comparing curve (M) with curve (G) of *Fig. 6.*

By the courtesy of Professor C. E. Inglis, O.B.E., M.A., LL.D., F.R.S., M. Inst. C.E., the Author was permitted to carry out some tests on the strength of columns of various types of teak coach-pillars. These tests were carried out on the 500-ton Amsler testing machine at the Engineering Laboratory, Cambridge University. The results of these tests are given in Table I. It will be noticed that a straight pillar is several times stronger than the ordinary curved coach-pillar, but the weakest of these pillars are strong enough for the load which they are called upon to carry in a railway coach under normal conditions.

The pillar-tests (A) to (M) show that the steel box-bracket fastening gives a much stronger pillar, and that the fracture in the latter case took place in the pillar itself, indicating that its full strength had been developed.

It is obvious from the above tests that a timber mortise-and-tenon joint with body-knees is not the most satisfactory method of securing timber pillars in railway-coach construction, but is the method handed down from early times by coach builders. If the timber body-framing of coaches is to be increased in strength, and the timber used to maximum advantage, it must be designed as an

TABLE I.

Type of pillar.	Dimensions of pillar.			Critical load: tons.	Type of Failure.
	Length: feet.	Width along centre line of coach: inches.	Thickness at right angles to centre-line of coach: inches.		
No. 1 Straight pillar	6	3	3	15·8	Fracture near centre of pillar.
No. 2 ,, ,,	6	3	3	18	Compression fracture near centre.
No. 3 ,, ,,	6	3	3	18·4	Compression fracture, shear on compression side.
No. 4 Standard pillar with mortise holes	6	3	3	8·6	Compression failure on inside of pillar near bottom mortise-hole and tensile failure at outside wall of mortise-hole.
No. 5 ,,	6	3	3	8·95	,,
No. 6 ,,	6	3	3	6·4	Failed by breaking up suddenly. Retained its load long time before failing.
No. 7 Curved pillar. No mortise holes	6	2	3	5·2	Large amount of deflection before failure. Failed by splitting along grain from bottom.
No. 8 ,,	6	2	3	4·4	Failed by splitting along the grain.
No. 9 ,,	6	$2\frac{1}{4}$	3	6·55	Compression failure on inside, 2 feet from bottom, tensile failure on outside.
No. 10 ,,	6	$2\frac{1}{4}$	3	6·15	Failed by shearing on inside and tensile failure on outside.
No. 11 Curved pillar with cross rebates	6	2	3	4·0	Failed by splitting from bottom cross rebate, and small compression failure on outside.
No. 12 ,,	6	2	3	5·55	Failed by splitting from bottom cross rebate.
No. 13 ,,	6	$2\frac{1}{4}$	3	5·75	Failed by splitting at bottom cross rebate; also compression failure on inside of bend.
No. 14 ,,	6	$2\frac{1}{4}$	3	5·7	,,
No. 15 ,,	6	3	$2\frac{1}{2}$	5·65	Failed by splitting downwards from bottom cross rebate.
No. 16 ,,	6	3	$2\frac{1}{2}$	5·0	,,

engineering structure on similar principles to a timber bridge, where timber is used for compression members and steel for fastenings, tension members, etc.

Design and Construction of a Welded Composite Vestibule-Coach.

The composite vestibule-coach which will be described below was built at the L.M.S. Rly. Co.'s Derby works, and was completed in April, 1934. This coach follows the standard practice of this Company in general exterior outline.

The principal dimensions of this coach are as follows:—

Length over body	57 feet 1 inch.
Width over body at waist	8 feet 11¼ inches.
Length over buffers	60 feet 8 inches.
Length of underframe	57 feet.
Centres of bogies	40 feet 6 inches.
Bogie wheelbase	9 feet.
Axle journals	9 inches by 4 inches.
Height from rail to top of roof-plates	12 feet 2½ inches.
First-class seats	18.
Third-class seats	18.
Tare weight	28 tons 16 cwts.

Fillet and Butt Welds.

Before commencing the design of the electrically-welded underframe and bogies, many full-size tests of welded joints were carried out, and the design was governed by the results of these tests. Taking into account the above tests and an allowance for impact and vibration, the Author decided upon the following values for the permissible unit stresses for calculating the strength of the welded underframe, bogies, etc.

Parent metal of mild steel 28 to 33 tons per square inch ultimate tensile strength.

Tension or compression	6 tons per square inch.
Shear	5 ,, ,,

Weld metal of high-grade covered electrodes used in conjunction with the above parent metal.

End fillets or tension	5 tons per square inch.
Compression	5·8 ,, ,,
Side fillets or shear	4 ,, ,,

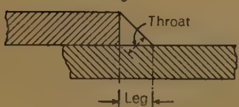
For convenience in making some calculations, these permissible stresses can be expressed in the form of Table II.

TABLE II.—SINGLE FILLET WELDS (*Fig. 7*).

"Leg" size of weld: inch.	"Throat" thickness: inch.	Permissible load per linear inch.			Weight of weld per linear inch: lb.
		Tension at 5 tons per square inch.	Compression at 5·8 tons per square inch.	Shear at 4 tons per square inch.	
$\frac{1}{8}$	0·088	0·44	0·51	0·35	0·002
$\frac{3}{16}$	0·133	0·66	0·77	0·53	0·005
$\frac{1}{4}$	0·177	0·88	1·03	0·71	0·009
$\frac{5}{16}$	0·221	1·11	1·28	0·88	0·014
$\frac{3}{8}$	0·265	1·33	1·54	1·06	0·020
$\frac{1}{2}$	0·354	1·77	2·05	1·41	0·035
$\frac{5}{8}$	0·442	2·21	2·56	1·77	0·055
$\frac{3}{4}$	0·530	2·65	3·08	2·12	0·080

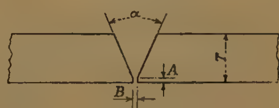
For double-fillet welds the above figures are multiplied by two.

Fig. 7.



FILLET WELD.

Fig. 8.



BUTT WELD.

The throat-thickness of butt welds (*Fig. 8*) is taken as the thickness "T" of the plate, or of the thinner plate if unequal, and the figures in Table II apply with the exception of those in the last column.

For butt welds up to $\frac{3}{16}$-inch plate there is no need for preparation, but a gap of about $\frac{1}{16}$ inch should be left between the plate-ends.

Suitable values for α, A and B are :—

Plates $\frac{1}{4}$ inch to $\frac{1}{2}$ inch, $\alpha = 70$ degrees, $A = \frac{1}{16}$ inch, $B = \frac{1}{16}$ inch.
Plates $\frac{1}{2}$ inch to $\frac{3}{4}$ inch, $\alpha = 60$ degrees, $A = \frac{1}{16}$ inch, $B = \frac{1}{8}$ inch.

Frequently it is preferable to use a "double-V" butt weld.

Design Notes.

(1) All welds to be calculated on the throat-thickness.

(2) Throat-thickness of a fillet weld is 0·707 of the leg dimension for equal leg fillets, and 0·707 of the shorter leg for unequal leg fillets, provided that concave-surface fillets are avoided.

(3) Sizes of all welds to be clearly marked on the drawings.

(4) Overhead and vertical welding to be avoided.

(5) Welds to be designed to require as little preparation as possible, on account of the cost of preparation.

(6) Small fillets are the most economical; doubling the throat-thickness of a weld increases the weld metal by four.

(7) Welded joints to be considered as being completely rigid and bending moments caused by this rigidity to be taken into account in the design.

(8) A margin of $\frac{1}{4}$ inch to $\frac{1}{2}$ inch to be allowed over the calculated length of welds, or a margin on the strength of the welding, to allow for the end craters.

(9) Combinations of side- and end-fillets to be used in preference to side- or end-fillets alone.

(10) As the savings to be effected by welding are made in the design office, the size of fillets, etc., should not be left to the welder to guess at, just as a riveter is not expected to guess at the size of rivets required in a bridge.

Calculation of the Sizes of Welds.

The method of calculating weld-sizes is as follows :—

(1) For a joint subject to tension, compression or shear.

$$\left.\begin{array}{l}\text{Safe load which can}\\ \quad\text{be transmitted by}\\ \quad\text{the welded joint}\end{array}\right\} = \left\{\begin{array}{l}\text{(Total length of welds)} \times \text{(throat-}\\ \quad\text{thickness of welds)} \times \text{(safe stress).}\end{array}\right.$$

(2) For a joint subject to a bending moment and a shear stress. Consider the example of *Figs. 9.* The cantilever of length l supports a load F, and produces a bending moment M. The moment of inertia I of the welds is obtained approximately by revolving the throat-thickness t of the welds into the plane of the joint as at (*b*), *Figs. 9.*

$$\begin{aligned}\text{Hence } I_{xx} &= \tfrac{1}{12}.bd^3\\ &= \tfrac{1}{12}[(m + 2t) \times (n + 2t)^3 - mn^3]\end{aligned}$$

The stress in the weld due to the bending moment is then given by the usual relationship

$$f = \frac{My}{I},$$

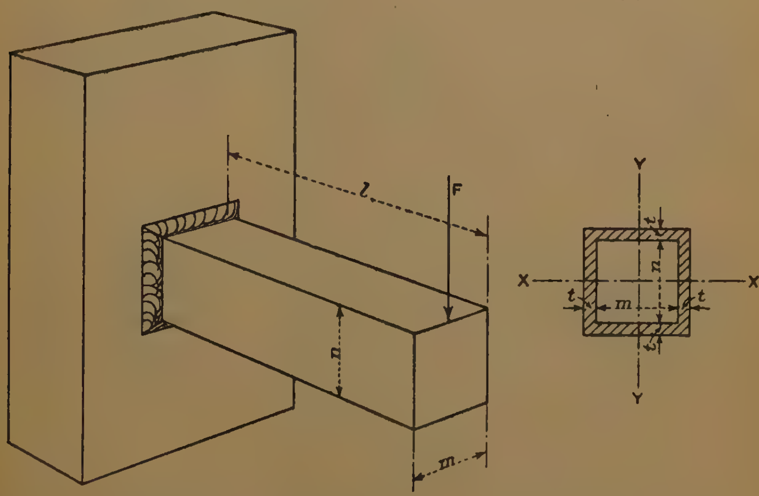
where f denotes the stress in the extreme fibre of the weld due to bending, and y denotes the distance from neutral axis to the extreme fibre of the weld.

The shear stress is calculated as in (1), the appropriate welds being taken into account, and from these values the maximum weld stress can be obtained, as in the example on p. 251.

It has been pointed out that it is very necessary that all weld sizes be calculated, and that the size of fillets, etc., be clearly marked on all drawings.

Figs. 9.

(*a*) (*b*)



JOINT SUBJECT TO BENDING MOMENT AND SHEAR STRESS.

Calculations of Strength of Welded Underframe.

The calculations of the strength of the welded underframe were based on the following loading :—

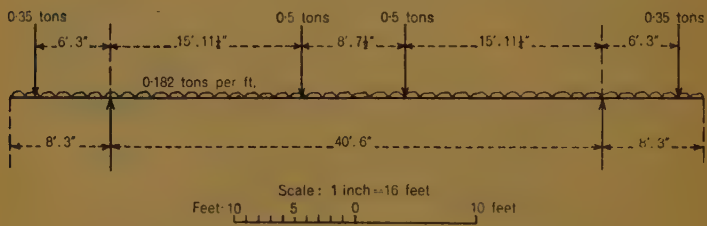
Weight of underframe	6 tons	14 cwts.	Evenly distributed.
,, ,, body	10 ,,	12 ,,	
,, ,, 50 passengers	3 ,,	3 ,,	
,, ,, hand luggage	0 ,,	6 ,,	
,, ,, rack luggage	0 ,,	14 ,,	Concentrated near ends.
,, ,, lavatory tanks	0 ,,	14 ,,	
,, ,, battery	1 ,,	0 ,,	Concentrated near queen-posts.
Total	23 tons	3 cwts.	

Total distributed load = 20 tons 15 cwts.
= 0·364 tons per foot over 57 feet.

It is assumed that half this load is taken on each solebar, that one solebar carries the electric lighting battery (1 ton), and that each carries a lavatory tank (0·35 ton) near one end and a luggage rack (0·35 ton) near the other. This gives the solebar considerably more load than it actually carries, but is on the safe side and simplifies the calculation. The solebar loading will then be as in *Fig. 10.*

It will be seen on referring to the bending-moment diagram of *Figs. 11* that 10-inch by $3\frac{5}{8}$-inch by $\frac{1}{2}$-inch channel solebars will carry the required loading of 20·75 tons. This is not necessarily the maximum safe load for the underframe as the longitudinals (9-inch by $3\frac{1}{16}$-inch by $\frac{3}{8}$-inch channels) in a welded structure will take a proportion of the load, although to some extent the diagonal bracing transfers the load to the solebars.

Fig. 10.



UNDERFRAME SOLEBAR LOADING.

From the bending-moment diagram the compression in the queen-post is 2·1 — 0·5 = 1·6 tons, where 0·5 ton denotes half the battery weight. If the queen-post is a 3-inch by 3-inch by $\frac{3}{8}$-inch angle the safe load is :—

$$= \frac{\frac{1}{5} \cdot A \cdot 21}{1 + \dfrac{l^2}{30{,}000 \times K^2}}$$

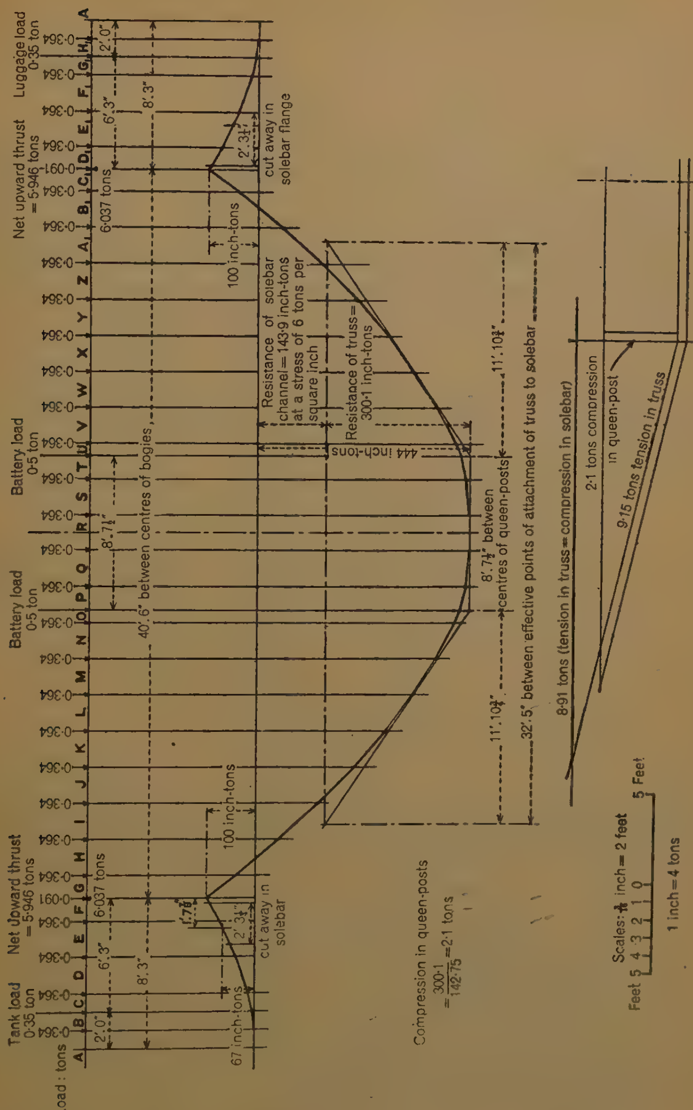
where l denotes the length in inches.
A „ area in square inches.
K „ least radius of gyration in inches.

$$= \frac{1}{5}\left[\frac{2{\cdot}11 \times 21}{1 + \dfrac{22^2}{30{,}000 \times {\cdot}581^2}}\right]$$

$= 8{\cdot}44$ tons.

The tension in the oblique truss member } = 9·15 tons (B.M. diagram).

Figs. 11.



UNDERFRAME BENDING-MOMENT DIAGRAM.

The area of the oblique truss member (3 inch by 3 inch by ½ inch L)	= 2·75 square inches.
The direct stress in member	= 3·33 tons per square inch.
The tension in the horizontal truss member	= 8·91 tons (*Figs. 11*).

The area of the horizontal truss member (3 inch by 3 inch by $\frac{1}{2}$ inch L) = 2·75 square inches.

The direct stress in member = 3·24 tons due to truss action.

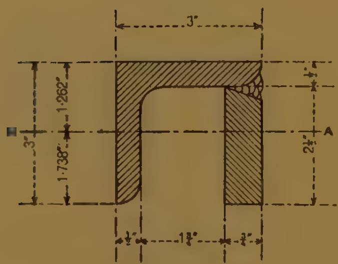
This member also carries half the weight of the battery box and cells weighing 1 ton.

Regarding this weight as evenly distributed over the 8-foot $5\frac{1}{2}$-inch length of this member, and regarding the member as simply supported at the ends :—

$$\text{Bending moment at centre} = \frac{0{\cdot}5 \times 101{\cdot}5}{8} \text{ inch-tons}$$

$$= \underline{6{\cdot}34 \text{ inch-tons.}}$$

Fig. 12.



Scale: one-quarter full size.

The section of this member is given in *Fig. 12*. The height of the neutral axis from bottom of section :—

$$= \frac{1{\cdot}5 \times 2{\cdot}75 + 2{\cdot}5 \times 1{\cdot}25 \times 1{\cdot}25}{1{\cdot}5 + 2{\cdot}5 \times 1{\cdot}25}$$

$$= \underline{1{\cdot}74 \text{ inch.}}$$

∴ Moment of inertia

$$= \tfrac{1}{3}\{3 \times 1{\cdot}26^3 - 1{\cdot}75 \times {\cdot}76^3\} + \tfrac{1}{3} \times 1{\cdot}25 \times 1{\cdot}74^3$$

$$= 3{\cdot}94 \text{ inch}^4 \text{ units.}$$

$$\text{Modulus of section} = \frac{3{\cdot}94}{1{\cdot}74} = 2{\cdot}27 \text{ inch}^3 \text{ units.}$$

The additional stress due to direct loading, assuming ends not fixed,

$$= \frac{6{\cdot}34}{2{\cdot}27} = 2{\cdot}79 \text{ tons per square inch.}$$

Hence the total stress due to "truss action" and direct loading from battery

$$= 2{\cdot}79 + 3{\cdot}24$$
$$\doteq \underline{6{\cdot}03 \text{ tons per square inch.}}$$

The assumption that the ends are not fixed is only partially true, but is made in order to estimate the greatest possible stress in the material.

In order to allow for the worst conditions in the joint between this member and the queen-posts, the truss was considered as perfectly fixed, and the maximum stress on the weld was found to be 3·88 tons per square inch.

Battery-box angle (3-inch by 3-inch by $\frac{3}{8}$-inch L).—This carries the other half of the weight of the battery-box and, as the amount of fixing (bolted connections) at the ends is not very effective, it may be regarded as simply supported :—

Modulus of section $= 0{\cdot}810$ inch³ units.

Bending moment $= \dfrac{0{\cdot}5 \times 101{\cdot}5}{8}$

$= 6{\cdot}34$ inch-tons.

$\therefore$ The maximum direct stress $= \dfrac{6{\cdot}34}{0{\cdot}810}$

$= 7{\cdot}80$ tons per square inch.

This member is the same as on the standard underframe, and as it has proved satisfactory during a number of years, it would appear that a factor of safety of about 4 is sufficient at this point.

Solebar section, 1 foot 7⅝ inches from bogie-centre; namely, where flange is cut away for doorway.—The design of body necessitated a widening of the underframe so as to bring the solebars beneath the body pillars. The underframe is 6¾ inches wider than the standard, but the width over the step-boards has not been altered. In order to obtain the same foot-room on the step when the door is open, it has been necessary to cut away the top flange of the solebar at the door-position, but the use of electric-arc welding has rendered it a fairly simple matter to reinforce this section by fixing a flange plate at the back of the solebar.

The bending moment at this point is resisted by the diagonal member A and the solebar section B shown in *Fig. 13* (p. 248).

Distance of neutral axis from bottom of bottom flange

$$= 4{\cdot}66 \text{ inches.}$$

Moment of inertia of solebar section

$$= \tfrac{1}{12}\{1{\cdot}25 \times 0{\cdot}56^3 + 2{\cdot}5 \times 0{\cdot}313^3 + 0{\cdot}49 \times 8{\cdot}88^3 + 3{\cdot}63 \times 0{\cdot}56^3\} + 0{\cdot}7 \times (9{\cdot}72 - 4{\cdot}66)^2 + 0{\cdot}781 \times (9{\cdot}59 - 4{\cdot}66)^2 + 4{\cdot}35 \times (5 - 4{\cdot}66)^2 + 2{\cdot}03 \times (4{\cdot}66 - 0{\cdot}28)^2$$

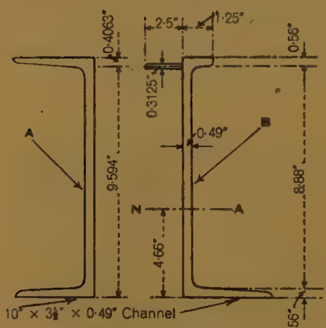
$$= \underline{104{\cdot}96 \text{ inch}^4 \text{ units.}}$$

Moment of inertia of diagonal section

$$= I \text{ for 10-inch by } 3\tfrac{1}{2}\text{-inch by } 0{\cdot}36 \text{ inch channel} + \frac{0{\cdot}125 \times 10^3}{12}$$

$$= \underline{119{\cdot}94 \text{ inch}^4 \text{ units.}}$$

Fig. 13.



Scale: one-eighth full size.

On the assumption that the diagonal and solebar deflect together, the moments of inertia may be added to obtain the total resistance,

$$= 119{\cdot}94 + 104{\cdot}96$$
$$= 224{\cdot}90 \text{ inch}^4 \text{ units.}$$

$$\text{The modulus} = \frac{224{\cdot}90}{5} = 44{\cdot}98 \text{ inch}^3 \text{ units.}$$

The bending moment at this section (*Figs. 11*) = 67 inch-tons.

$$\therefore \text{ Maximum fibre stress} = \frac{67}{44{\cdot}98} = \underline{1{\cdot}49 \text{ ton per square inch.}}$$

If the solebar alone resists the bending moment,

$$\text{then maximum fibre stress} = \frac{67 \times 5{\cdot}34}{104{\cdot}96} = \underline{3{\cdot}40 \text{ tons per square inch.}}$$

Shear stress at the bottom joint of the flange plate to the web of

the solebar was calculated by the Author as follows, as no standard method appears to be in existence for calculating such welds :—

$$q = \frac{F}{IZ}\int_{y}^{y_1} yzdy$$

At this section, the shear force $F = 0{\cdot}182 \times 6{\cdot}59 + 0{\cdot}35 = 1{\cdot}55$ tons.

$$q = \frac{1{\cdot}55}{104{\cdot}96 \times 2{\cdot}99}\{1{\cdot}25 \times 0{\cdot}56 \times 5{\cdot}06 + 2{\cdot}5 \times 0{\cdot}313 \times 4{\cdot}93 + 0{\cdot}49 \times 0{\cdot}003 \times 4{\cdot}78\}$$

$= 0{\cdot}04$ ton per square inch.

$\therefore$ The approximate force exerted per inch of weld-fillet

$$= \frac{2{\cdot}5}{2} \times 0{\cdot}04 \text{ ton.}$$

$$= 0{\cdot}05 \text{ ton approx.}$$

Thus $\frac{1}{8}$-inch fillet welding will be satisfactory.

Centre-casting crossbars.—Regarding the crossbars as supported at the centre and carrying equal loads at the ends of 5·79 tons, and of length 7 feet 10$\frac{1}{4}$ inches,

Maximum bending moment $= 5{\cdot}79 \times 47{\cdot}125$ inch-tons
$= 273$ inch-tons.

Using two 10-inch by 3$\frac{5}{8}$-inch by 0·49-inch channels,

Modulus of section $= \frac{119{\cdot}94 \times 2}{5}$ inch units.
$= 47{\cdot}98$ inch units.

$\therefore$ Maximum fibre stresses $= \frac{273}{47{\cdot}98}$ tons per square inch.
$= 5{\cdot}69$ tons per square inch.

Space will not permit the reproduction of all the calculations, but sufficient has been given to indicate the methods adopted.

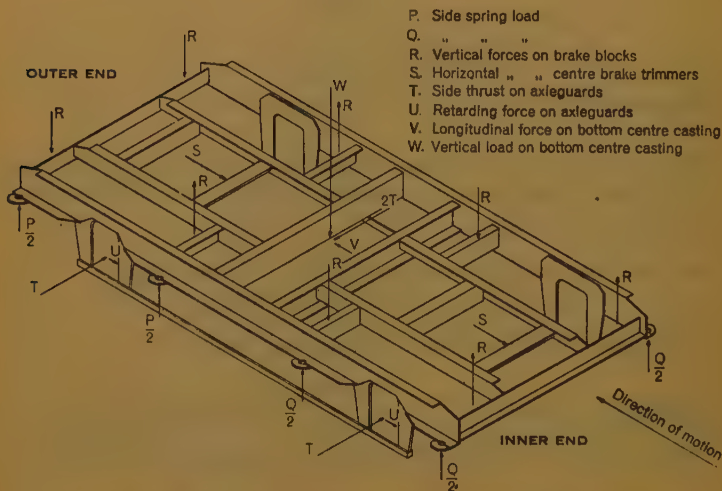
Welded joints of the underframe.—All the important joints of the underframe members have been calculated for welding by the methods which have been previously explained, and examples will be given later. The welded joints are in all cases stronger than the corresponding joints of a standard riveted underframe. The design of the welded underframe is shown in detail in Figs. 14, Plate 1. It will be noticed that the size of fillet welds is given in terms of the

throat-dimensions. This method was adopted by the Author prior to the publication in March, 1934, of the British Standards Specification for Metal Arc Welding (No. 538), after which welds have been specified by their leg-dimensions, although it is of little importance which method of specifying the welds is used as it does not influence the result of the calculations.

Calculations for the Welded Bogies.

The bogies were first calculated for a loading of 11 tons 11 cwts.; this was increased to 12 tons 12 cwts., as it was thought that the

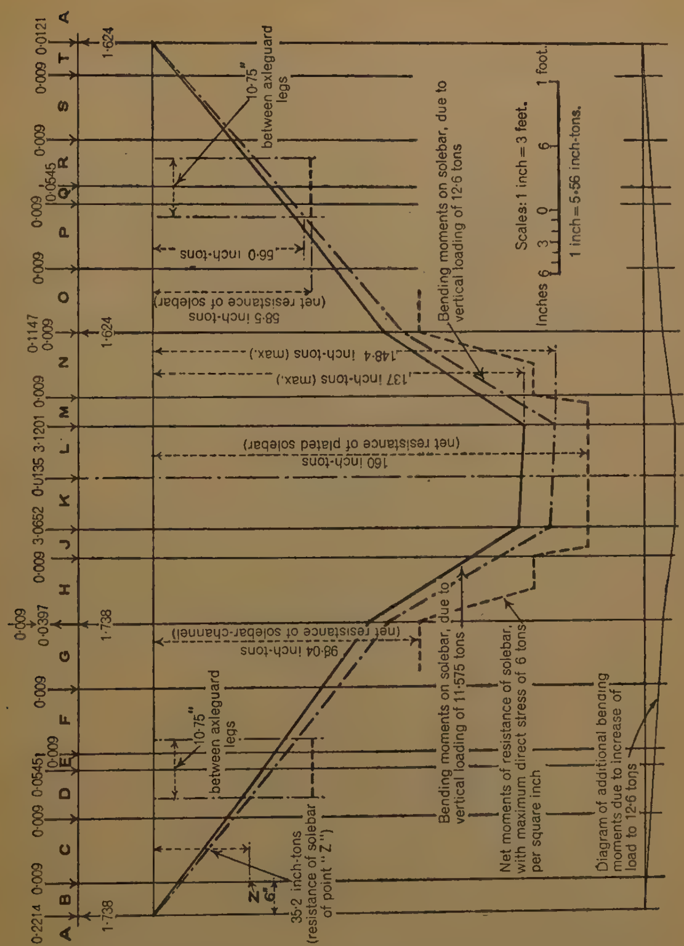
Fig. 15.



Positions of Principal External Forces on Bogie Frame.

tare weight of the coach might exceed the original estimate of 29 tons and be as much as 31 tons. In the calculations various sources of stress were taken into account, and the weight of the bogie frame was distributed so that the weight as far as possible was allowed for at the point where it occurred in practice. The positions of the principal external forces on the bogie frame are shown in *Fig. 15*, and the bending-moment diagram for the bogie solebars in *Figs. 16*. The shear values have been calculated for the underframe and bogies where necessary, and are not shown as shear diagrams. The maximum calculated direct stress does not exceed 6 tons per square inch for any part of the bogie frame with a load of 12 tons 12 cwts.

Fig. 16



BENDING-MOMENT DIAGRAM FOR BOGIE SOLEBARS.

Some of the stresses in the principal members are :—

Item.	Maximum direct stress.	Remarks.
Bolster	5·03 tons per square inch	Tension.
Swing beam	4·10 ,, ,,	,,
Bogie crossbar . . .	5·14 ,, ,,	,,
Solebar	5·98 ,, ,,	Between axle-guard legs.

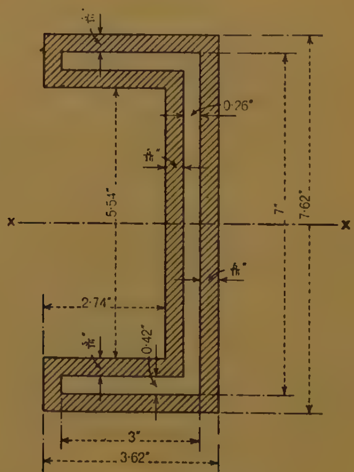
Calculations of the welding of the joints.—As an example of the method of calculating the size of welds, consider a joint similar to

the crossbar connection to a solebar on the bogie, but having fixed ends, and assume for the purpose of this example the following vertical loading :—

(*a*) Shear of 12 tons.
(*b*) Bending moment of 25 inch-tons.

It is proposed to weld all around the end of the 7-inch by 3-inch channel with a $\frac{5}{16}$-inch throat fillet. It will be necessary first to find the moment of inertia of the shaded section of *Fig. 17*, which is obtained by revolving the throat thickness of the weld into the plane of the joint and neglecting the radii.

Fig. 17.



Scale: one-quarter full size.

The moment of inertia, I_{xx},

$$= \tfrac{1}{12}(3{\cdot}62 \times 7{\cdot}62^3) - \tfrac{1}{12}(2{\cdot}74 \times 5{\cdot}54^3) - 32{\cdot}75$$
$$= 133{\cdot}47 - 38{\cdot}82 - 32{\cdot}75$$
$$= 61{\cdot}90 \text{ inch}^4 \text{ units.}$$

The stress f_1 due to the shear taken by the web welds is given by :—

$$f_1 = \frac{12}{2 \times 5{\cdot}54 \times 0{\cdot}31} = 3{\cdot}49 \text{ tons per square inch.}$$

The stress f_2 due to the bending moment is given by :—

$f_2 = \dfrac{M}{Z}$, where M denotes the bending moment
and Z ,, section modulus of the welds.

Now $Z = \dfrac{61{\cdot}90}{3{\cdot}81} = 16{\cdot}25$ inch³ units.

$\therefore \quad f_2 = \dfrac{25{\cdot}0}{16{\cdot}25} = 1{\cdot}54$ ton per square inch.

The maximum weld stress f according to Continental practice is given by :—

$$\begin{aligned} f &= \sqrt{f_1^2 + f_2^2} \\ &= \sqrt{3{\cdot}49^2 + 1{\cdot}54^2} \\ &= 3{\cdot}81 \text{ tons per square inch.} \end{aligned}$$

This maximum stress must not exceed the permissible shear stress of 4 tons per square inch given for weld metal on p. 240. The above method of calculating weld sizes agrees reasonably well with practical tests, as shown in the Appendix.

In cases like that shown in *Fig. 17* it can be assumed that the shear will be almost exclusively transmitted by the web welds, and the direct stresses due to bending by the flange welds.

If it can also be assumed that both shearing and direct stresses exist at a point on a known plane, the Author is of the opinion that a more accurate determination of the actual maximum stresses can be arrived at in the following manner :—

The moment of inertia for the flange welds only, I_{xx},

$$\begin{aligned} &= 61{\cdot}90 - \tfrac{1}{12}(0{\cdot}63 \times 5{\cdot}54^3) \\ &= 53{\cdot}0 \text{ inch}^4 \text{ units.} \end{aligned}$$

Hence, $\quad Z = \dfrac{53{\cdot}0}{3{\cdot}81} = 13{\cdot}91$ inch³ units.

$\therefore f_2 = \dfrac{25{\cdot}0}{13{\cdot}91} = 1{\cdot}80$ ton per square inch.

and $\quad f_1 = 3{\cdot}49$ tons per square inch as above.

Then the maximum principal stress

$$\begin{aligned} &= \frac{f_2}{2} + \sqrt{\frac{f_2^2}{4} + f_1^2} \leqq 5 \text{ tons per square inch (p. 240).} \\ &= \frac{1{\cdot}80}{2} + \sqrt{\frac{1{\cdot}80^2}{4} + 3{\cdot}49^2} \\ &= 4{\cdot}5 \text{ tons per square inch.} \end{aligned}$$

The maximum shear stress

$$\begin{aligned} &= \sqrt{\frac{f_2^2}{4} + f_1^2} \leqq 4 \text{ tons per square inch (p. 240).} \\ &= \sqrt{\frac{1{\cdot}80^2}{4} + 3{\cdot}49^2} \\ &= 3{\cdot}6 \text{ tons per square inch.} \end{aligned}$$

From the above it will be seen that a $\frac{5}{16}$-inch fillet weld will satisfy the given vertical-loading conditions about the XX axis, although both methods of calculating welds must be regarded as approximate, owing to the several assumptions made.

The design of the welded bogies is shown in Figs. 18, Plate 2. It follows the L.M.S.R. standard practice in general lines, being built up from rolled-steel sections, but all the framing members are arc-welded together, and other members such as the bolster and swing beam are built up from welded parts. The only parts fixed by riveting or bolting are those which require fairly frequent removal, such as brake-work, axle-guard liners, etc. Due to the elimination of rivet-holes, it has been found possible to use lighter sections in several places, notably the solebars. These have been reduced considerably in weight, and reinforced where necessary by flange plates. Web-stiffeners have also been welded in position at loading points.

As in the case of the bogie, it has been found possible in the underframe design to reduce a few of the sections owing to the elimination of rivet-holes. Care has been required, however, as the underframe details are subjected to more indeterminate loading than the bogie framing. There is a large field for the use of electrically-welded built-up parts in place of steel castings, and an example of this is seen in the buffer-trimmer and knee casting, which has been replaced in this underframe by plates welded in position.

The Construction of the Welded Bogies.

Before commencing with the construction of the bogies, a welding-jig for the bogie frames was constructed. This jig was made of rolled-steel sections welded together to form a framework-table standing on six legs. Short lengths of angles were welded at suitable places in order to position the various members of the bogie frame, and a number of taper pins were secured in the jig so as to set the position of the side spring-plates. The jig with the bogie frame in position is shown in *Fig. 19* (facing p. 235).

The various stages in the construction of the welded bogies were as follows :—

(1) All steelwork was prepared to the correct length, and the ends milled, etc., as shown on the drawings.

(2) All small components were welded to the main members, such as brake-hanger brackets to the headstock and brake-trimmers.

(3) Axle-guards, web-stiffeners, spring-brackets, and flange-plates were welded to the solebars.

(4) Sections and plates for the bogie bolster and the spring beam were placed in a jig and welded.

(5) The solebars, headstocks, crossbars, etc., were placed in their correct relative positions in the jig, and tack-welds made.

(6) Gusset-plates were tacked in position.

(7) All welds which could be done in a "downhand" manner were executed, starting from inside the frame and gradually working outwards, doing the diagonally opposite joints of the headstock and solebar in rotation last of all.

(8) The frame was turned upside down, and the welds completed in a similar order to (7) above.

(9) The bogie frame was then turned on one side and the welds completed.

(10) The frame was finally turned on to the opposite side to (9) above, and the remainder of the welding finished.

It will be noticed that no overhead or vertical welding was done in the above procedure, as the Author considered it better to turn the frame into the easiest position for welding than to risk any inferior welding. It was not found necessary to straighten any part of the bogie frame after welding, except the solebar after operation (3) above.

A high-grade covered electrode and an A.C. welding machine were used throughout for the welding of the bogies and underframe. The physical properties of the electrode were :—

Tensile strength	30–32 tons per square inch.
Elongation on 4 diameters	25–28 per cent.
Elongation on 2 diameters	30–35 ,,
Reduction in area	35–38 ,,
Izod value	45–50 foot-lbs.
Brinell number	130–140

All the welds were carefully inspected and in a few cases welds were chiselled out and done again. The welds were also tested with a welding gauge to ascertain whether the throat-dimensions were up to those specified on the drawings.

The following particulars were recorded on welding record-sheets for all welds :—

(1) Job or joint number of plan or drawing.
(2) Type of electrode used.
(3) Gauge of electrodes used.
(4) Number of electrodes used.
(5) Inches of electrode wasted.
(6) Inches of welding done.
(7) Number of runs of welding.
(8) Power consumed.

(9) Current in amperes.
(10) Time taken in welding.
(11) Time taken in setting up and marking out.
(12) Size of finished weld.
(13) Welding inspected by —.
(14) Crane and lifting time.
(15) Name of welder.
(16) Remarks.

All the above information was recorded for the first coach only.

The Construction of the Welded Underframe.

The coach underframe was welded in a cradle or jig consisting of a number of lengths of 9-inch by $3\frac{1}{16}$-inch channel placed parallel at suitable intervals on low trestles and connected on the outsides by two lengths of 10-inch by $3\frac{5}{8}$-inch channel, 58 feet long. Short lengths of angle were welded at convenient places to position the underframe members.

The underframe was constructed in the following order :—

(1) All steelwork was prepared to correct length, etc., as shown in the drawings, except the solebars, which were left slightly longer.

(2) $\frac{1}{2}$ inch camber was put into the solebars.

(3) Trussbars and struts were welded to the solebars, and struts to the longitudinals.

(4) Both sets of crossbars had top and bottom plates welded on, all components were welded to main members, and buffer trimmers, etc., were prepared.

(5) Solebars with trussbars attached were placed in the jig.

(6) Longitudinals and crossbars were placed in the jig and tack-welded.

(7) The remainder of the truss framing was placed in position and tack-welded.

(8) Diagonals and gussets were tacked in position.

(9) All horizontal welds were made, starting from the inside and working outwards.

(10) Buffer trimmer-plates were welded in position.

(11) Diagonal and cross bracing was welded, starting between the longitudinals.

(12) Solebars were cut to correct length, and the headstocks welded in position.

(13) All horizontal welds were completed.

(14) The jig was completely removed from the underframe.

(15) The underframe was turned right side up by means of the crane.

(16) All horizontal fillets were completed.

(17) The underframe was turned on its left side and welded.

(18) The underframe was turned on its right side and the welding completed.

(19) The underframe was turned the correct side up.

(20) The solebar-flanges were cut for the doorway clearances.

(21) The step-board brackets were welded to the solebars.

A welding record was kept for the underframe similar to that for the bogies.

The largest welds on the bogies or underframe were $\frac{5}{16}$-inch throat fillets and were used for main-frame connections ; $\frac{1}{4}$-inch and $\frac{3}{16}$-inch welds were used for other joints.

Great care was exercised in the preparation and welding of the underframe members to prevent distortion and shrinkage. This consideration was well rewarded by the fact that the maximum distortion at any point amounted to a total shrinkage in width of $\frac{5}{16}$ inch. Other total shrinkages in width along the underframe varied from $\frac{1}{32}$ inch to $\frac{1}{8}$ inch. These shrinkages are negligible as the lining-up of the body pillar-brackets eliminated this slight variation in width. The underframe required no straightening of any kind after the welding, and was as true and straight as any riveted underframe. The fitting of wheels, axle-boxes, brakework, steam-warming and vacuum pipes, etc., to the bogies and underframe will not be described as they followed the usual procedure.

The completed welded underframe is shown in *Fig. 20* (facing p. 235).

The Welders.

It is very important that all men engaged on welding should be adequately trained, and their work should be tested at regular intervals by means of test bars. All the welders engaged on the work described in this Paper had to execute a number of tests before being allowed to start. These tests involved the welding of full-size joints of rolled-steel sections, as well as the more usual butt- and fillet-weld tests where the welds are in tension, shear, etc. The results of these tests were recorded, as well as those of an alternating-stress test which each welder performed monthly.

Test of the Welded Underframe and Bogies.

Loading and Deflection Test.—The underframe mounted on its bogies was placed on a level piece of track near the stock-yard and

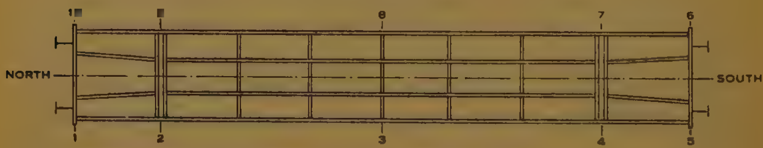
loaded, and the deflections measured. It was estimated that the following distributed load would be required :—

To represent the coach body	. . . .	14 tons.
,, ,, battery and dynamo	. . . .	1 ,,
,, ,, passengers and luggage, plus 100 per cent. overload	. . . .	8 ,,
		23 tons.

To represent the lavatory tanks filled with water, it was decided to put at each end of the underframe a load of 650 lbs. with its centre of gravity 2 feet from the headstock, and 2 feet 6 inches from one solebar.

The load was made up of lengths of 9-inch by 3-inch and 12-inch by $3\frac{3}{4}$-inch channels laid across the underframe at suitable points.

Fig. 21.



OBSERVATION POSITIONS FOR LOADING AND DEFLECTION TEST.

When the above loading was complete, the underframe and bogies were weighed and the weights at the two bogies were found to be :—

North end	. . . .	18 tons 19 cwts. 2 qrs.
South end	. . . .	19 ,, 0 ,, 3 ,,
Total	. . . .	38 tons 0 cwts. 1 qrs.

The underframe was returned to its original loading position and short pieces of angle were placed on the ground beneath the underframe solebars at the ends, middle, and crossbars, to form datum levels from which the heights of the underside of the solebars were marked off on staffs. Before marking off, the staffs were carefully set vertical and their positions marked so as to ensure exact repetition of these positions for each observation of height.

In addition to the observations on the underframe solebar, an 8-foot straight edge was placed in a central position along the top of each solebar, supported at the ends on small packing pieces so as to avoid the central flange-plates. The distance at the centre

between the top of this solebar top plate and the straight edge was carefully observed for each side of each bogie. These observations were repeated after the load had been reduced by an amount estimated at 8 tons, and again after the load had been entirely removed. The plan of observation positions is given in *Fig. 21.* The observations obtained were as follows :—

UNDERFRAME DEFLECTIONS.

West side.				East side.			
Staff No.	Deflection between			Staff No.	Deflection between		
	No load and full load : inch.	No load and light load : inch.	Light and full load : inch.		No load and full load : inch.	No load and light load : inch.	Light and full load : inch.
Centre 3	0·51	0·31	0·21	Centre 8	0·53	0·32	0·20
South end 5	– 0·29	– 0·18	– 0·10	South end 6	– 0·35	– 0·27	– 0·08
North end 1	– 0·30	– 0·23	– 0·06	North end 10	– 0·32	– 0·20	– 0·13
Mean central deflection . . .					0·52	0·315	0·205
Mean end deflection					– 0·31	– 0·22	–0·09

BOGIE-SOLEBAR DEFLECTIONS.		Deflection between		
		No load and full load : inch.	No load and light load : inch.	Light and full load : inch.
North bogie	West side . . .	0·04	0·04	0·02
	East side. . . .	0·05	0·03	0·02
South bogie	West side . . .	0·05	0·03	0·02
	East side. . . .	0·05	0·03	0·03
Mean of bogie-solebar deflections over central 8-foot span . . .		0·05	0·03	0·02

Shunting Test.—The underframe, mounted on its bogies, was shunted into two stationary 20-ton goods brake-vans with locked wheels at a speed of 9·5 miles per hour. The speed was measured with a stop-watch over a taped distance of 100 feet. The buffer castings on the first goods brake-van were broken, but examination of the welded underframe and bogies revealed no damage.

Construction of the Coach-Body.

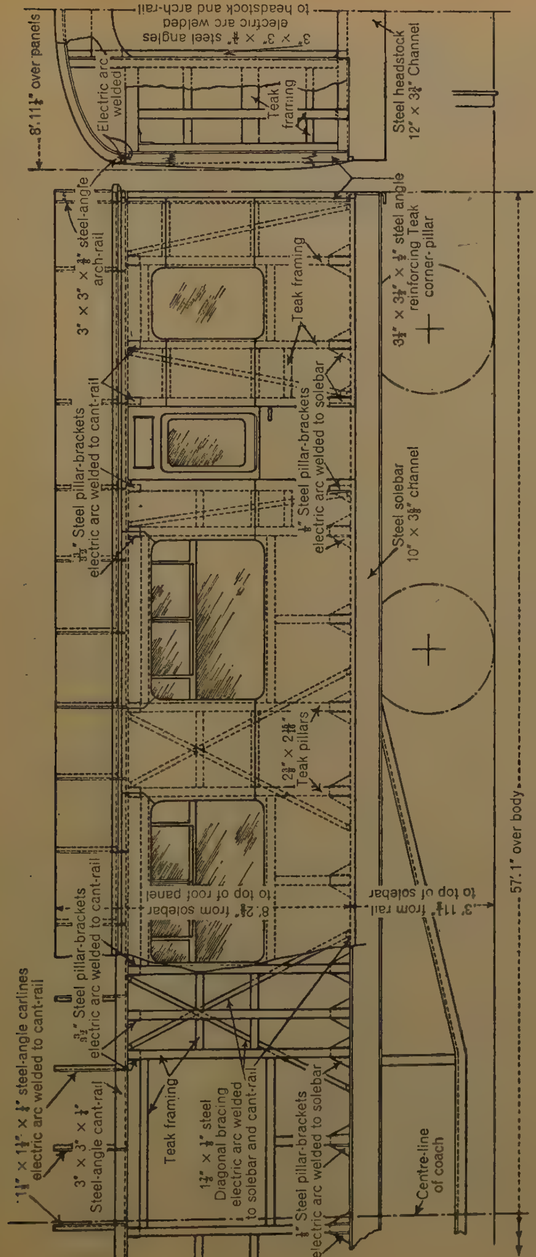
The results from the investigation of the body-framing of passenger coaches, already described, were incorporated into a new design of body-framing, which is shown in *Figs. 22* and *23* and is covered by British Patent No. 418428. This design of body necessitated a $6\frac{3}{4}$-inch widening of the underframe in order to bring the solebars beneath the body-pillars.

The tests proved that the usual mortise-and-tenon joint, reinforced with a knee-bracket, is not the most satisfactory method of fixing the pillars. The mortise-and-tenon joint fails long before the full strength of the pillar has been developed, and a new fastening in the form of a steel box-bracket obviates this defect. The solebar box-brackets are shaped to form three sides of a box with gussets at two sides. These were pressed from $\frac{1}{8}$-inch plate steel, and were welded directly to the underframe solebar, as shown in *Figs. 22* and *Fig. 4* (centre). The solebar box-brackets receive the plain cross-cut ends of the teak pillars, and are secured by two $\frac{3}{8}$-inch diameter countersunk bolts.

The use of the welded box-bracket type of joint enables the designer to reduce the section of the body side-pillars considerably. In this coach the reduction in cross-sectional area has been about 21 per cent.; in addition it was found possible to eliminate about one hundred subsidiary members without weakening the finished framing. (Curves D and A, *Fig. 1*, and curves E and A, *Fig. 5*.) As already mentioned, the side-pillars stand directly on the underframe-solebars, and no timber bottom-side and floor framing has been provided. In this manner a considerable saving in weight has been effected, and the connection between the body and underframe has been made more rigid and shock-proof. The flush top-finish of the underframe which electric welding has made possible has facilitated the direct fixing of the steel key-sheeting. The key-sheeting is secured to the underframe members by means of steel washers welded to the frame through holes drilled in the key-sheeting. In the formation of the floor this key-sheeting is covered with cork slabs shaped to fit into the grooves and cemented in position with bituminous solution. This forms a base for the felt and linoleum floor finish.

The ends of the coach are built up of steel angles welded together and to the headstocks. Two of these $3\frac{1}{2}$-inch by $3\frac{1}{2}$-inch by $\frac{1}{2}$-inch steel angles form very strong corner-pillars, and the ends of the steel cant-rails are welded into the joints made between them and the steel arch-rail of 3-inch by 3-inch by $\frac{3}{8}$-inch angles. The 3-inch by 3-inch by $\frac{1}{4}$-inch steel-angle cant-rail is several times stronger than the wooden cant-rail which it displaces. Steel box-brackets, similar

Figs. 22.

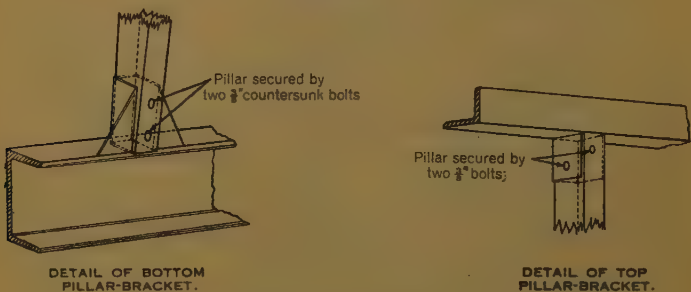


New Arrangement of Body Framing in Composite Vestibule-Coach.

in principle to those already described but made from $\frac{3}{32}$-inch thick steel, are welded to the underside of the cant-rails and form the top joints of the side-pillars.

A source of weakness in the standard timber end-framing has been the gangway back-angle, which was bent round immediately above the gangway sheet. Any shock which resulted in excessive pressure on to the gangway was liable to cause this back angle to penetrate the timber end-framing. This member has been eliminated in the new design, and the gangway sheet is fastened to two 3-inch by 3-inch by $\frac{3}{8}$-inch steel angles which form the gangway door-pillars for the end. These extend from the headstock to the steel arch-rail, and are welded in position, thus forming a strong support. Teak skeleton framing is bolted to the steel end-pillars, and acts as a support for the steel panelling, etc. It can be claimed that the strength of the end of this coach will be about equal to that of a steel coach.

Figs. 23.



ARRANGEMENT OF PILLAR-BRACKETS.

The cant-rails are drilled for riveting the steel roof sheets, side panels and gutter sections to them. The steel carlines of $1\frac{1}{2}$-inch by $1\frac{1}{2}$-inch by $\frac{1}{4}$-inch angles are welded at intervals to the cant-rails. Diagonal bracing has been introduced into the body side-framing to increase its strength and shock-resisting properties. The diagonal bracing is of $1\frac{7}{8}$-inch by $\frac{1}{8}$-inch section steel, welded between the solebar and cant-rail. It is designed to offer no resistance to compression, so that there is no tendency to put a tension on the timber pillars and fastenings. All cross battens, window sills, etc., are mortised and tenoned into the pillars.

The procedure in erecting the body-framing was as follows :—

(1) All timber pillars, cross battens, steel corner- and gangway-angles, steel cant-rails, arch-rails, pillar box-brackets, etc., were prepared according to the drawings. The underframe and bogies were placed in position, and levelled and packed.

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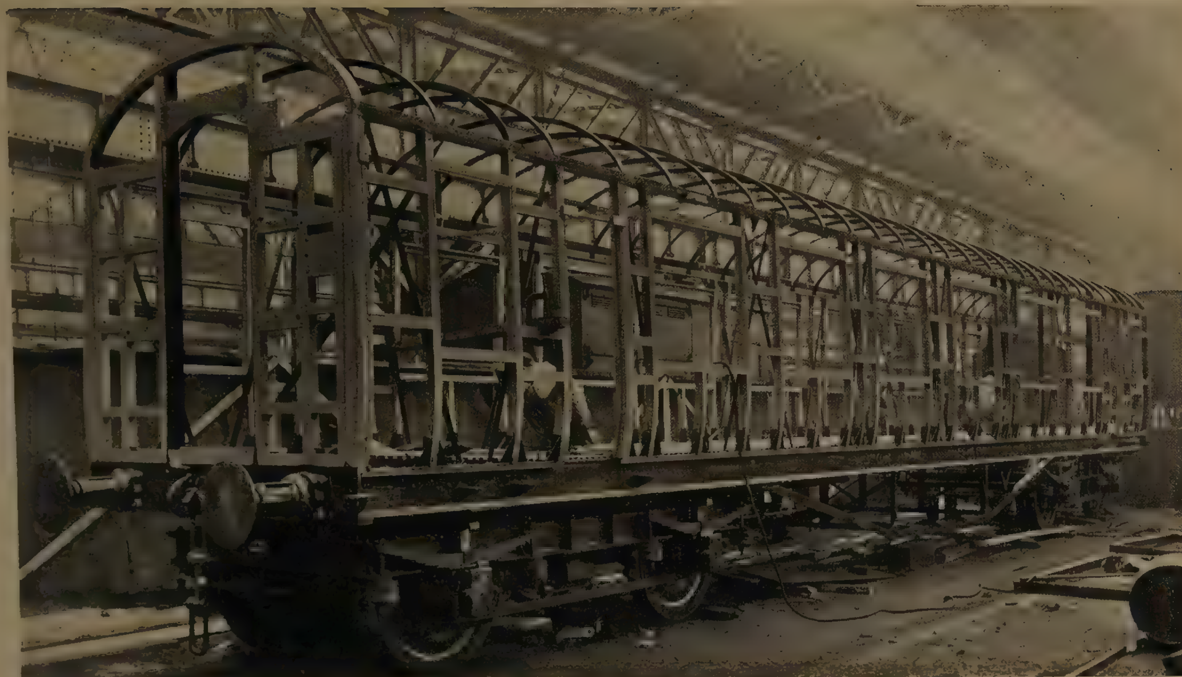
Fig. 24.



COMPLETED BODY-FRAMING.

Fig. 26.

COMPLETED COACH.

Fig. 37.

COMPLETED WAGON.

(2) Solebar and cant-rail box-brackets were put on to the pillars and secured with bolts. (It is very important that these box-brackets should be a tight fit on the pillars, especially in the case of the solebar box-brackets.)

(3) The body framing was assembled in " quarters."

(4) The four steel corner angles and the four gangway-angles were welded to the headstocks.

(5) The body side-framing was placed in position on the solebars and lined up, and temporarily secured by timber supports in correct alignment, whilst the solebar box-brackets were welded to the solebars.

(6) The steel arch-rails were welded to the steel cant-rails in sections about one-third the length of the coach, the cant-rails being held in a jig.

(7) The cant-rails with arch-rails attached were then placed on the pillars in three sections, and when in correct alignment, the cant-rail box-brackets were tack-welded to the cant-rails.

(8) The cant-rails with arch-rails and box-brackets attached were removed from the coach, and placed upside down on trestles, thus enabling the welding of the box-brackets to be completed in a horizontal position.

(9) The cant-rails with arch-rails and box-brackets attached were next placed in their final position on the pillars, and secured with bolts. The transverse bolt-heads were welded to the box-brackets through drilled holes as shown in *Figs. 23*. The advantages of this method of fastening are, firstly, a flush finish on the inside of the box-bracket, and, secondly, the fact that the pillar can be removed from the bracket if necessary.

(10) The joints in the sections of the cant-rails were butt-welded.

(11) The gangway and corner-angles were welded to the end arch-rails and to the cant-rails. These eight welds were practically the only overhead welds on this coach.

(12) The diagonal braces were welded in position, being left about $\frac{1}{16}$-inch short in length so that a slight tension was put into them after the welding had cooled.

(13) The steel key-sheeting for the floor was marked out, laid on the underframe in its correct position, and then welded to the underframe through $\frac{1}{2}$-inch washers and $\frac{5}{8}$-inch holes drilled in the galvanized key-sheeting.

The completed body-framing is shown in *Fig. 24* (facing p. 262). Space does not permit of a detailed description of the finishing of the exterior and interior of the coach, hence only points of special interest will be mentioned.

Body Details.—The vestibule partitions were fixed to the key-sheeted floor by means of small troughs made from No. 16 S.W.G. metal and riveted to the key-sheeting. At the intersection of two diagonal braces, small pieces of felt were fixed to prevent any noise due to vibration. Timber battens for securing the seat ends were laid flush with the cork flooring and bolted to the key-sheeting.

Inside the compartments, fixed lights are fitted to a height of 2 feet above the waist-rail. These are 4 feet 6 inches long in the first class, and 4 feet long in the third class. Above these a further 1 foot is occupied by metal-frame sliding lights, which slide back on either side into pockets in the coach-framing, giving a maximum opening area of 3·2 square feet in the first class, and 3 square feet in the third class. Both the fixed and sliding lights are fitted with "Armourplate" glass, which has superior shock-resisting properties, and would prove less dangerous than ordinary glass in the event of a blow from any cause. If broken by an excessive blow the glass shatters into very small fragments which are too small to cause any greater injury than surface-scratches.

In addition to the ventilation obtainable by the sliding lights, roof-ventilation is provided by air-extractors. Four extra roof-ventilators are fitted which communicate with the air-space between the outer panelling and the inner lining of the coach. Provision is made for the passage of air throughout these spaces from the bottom of the coach-sides where there is direct communication with the atmosphere, up to the roof where it is extracted by the ventilators. It is anticipated that this will minimize the formation of condensation and any consequent corrosion.

The roof panels are made of No. 16 S.W.G. galvanized steel, and are all 4 feet 0⅝ inch in length, except the two end panels which are 4 feet 4¼ inches. The body side- and end-panels are of No. 16 S.W.G. steel having a superior charcoal finish. The panels are secured to the body- and roof-framing members by sherardized wood-screws, the holes for which are counter-sunk punched. Grooves to accommodate the projections of the punched holes are milled in the timber framing where required.

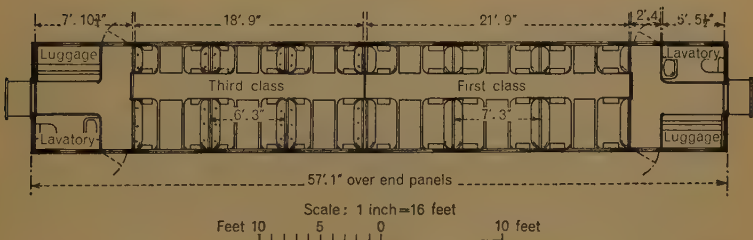
Interior Details.—The interior of this coach consists of two of the usual open or vestibule-type compartments, each providing seating- and table-accommodation for eighteen passengers. The first-class compartment is 21 feet 9 inches in length, and the third class 18 feet 9 inches. In addition to the vestibules, lobbies at each end provide luggage- and lavatory-accommodation, as well as the means of entry and exit (*Fig. 25*).

Throughout the design of the interior, every endeavour has been made to eliminate unnecessary curves and ornamentation. The

result is a simple and somewhat severe arrangement which accords well with the modern trend of domestic design without sacrificing comfort or convenience. Following this general motif, the standard elliptical contour of ceiling has been abandoned for one which incorporates plain surfaces.

The metal fittings throughout the first-class end are chromium-plated, and are designed to follow the straight-line motif. This is also expressed in the quartered veneering of Circassian walnut on three-ply which is used for the interior finish, and the compartment sliding-doors. To harmonise with the doors, two frameless rose-tint mirrors are fitted on either side, with simple landscape designs sandblasted at the back of the glass so that the lines of the design radiate away from the doors in each case. The curtain boxes are fitted with tubular lamps controlled by switches on the light-rails,

Fig. 25.



PLAN OF COMPOSITE VESTIBULE COACH, SHOWING SEATING ARRANGEMENT.

and these will take the place of the usual table lamps. The general lighting is provided by box fittings in the ceilings.

The seats are upholstered in cut velvet moquette. They have been constructed to afford increased comfort, and are spaced 7 feet 3 inches apart in the first class, and 6 feet 3 inches in the third class. The seat as a whole harmonizes with the finish of the rest of the compartment. It was found possible to effect a saving in weight for the seating to the extent of 5 cwts. for the coach. Part of this saving in weight is from the seat-end which is a framed structure finished with veneered three-ply. In addition to the linoleum and felt above the cork flooring, there is a specially-designed carpet in the first-class compartment.

The third-class interior is finished in polished mahogany veneered on three-ply, with upholstery of green and brown uncut moquette. The fittings are in Venetian bronze, and the whole design carries out as far as possible the straight-line motif of the rest of the coach.

Westinghouse steam heaters are installed in the two vestibules, and hot water heaters in the lavatories. The electric lighting system in this welded coach is the standard of the L.M.S. Rly. Co. known as the Wolverton system. The completed coach is shown in *Fig. 26* (facing p. 263).

Weights.—The advantage of electric-arc welding applied to railway-coach construction is the reduction in weight which can be effected. This reduction in weight is obtained owing to the elimination of rivet-holes, etc., and the higher efficiency of welded joints compared with riveted joints. The reductions in weights which have been obtained on this welded coach are as follows :—

	Standard coach.			Welded coach.			Saving.	
	Tons	cwts.	qrs.	Tons	cwts.	qrs.	Cwts.	qrs.
Ordinary bogie including wheels, axleboxes, etc.	5	3	2	5	0	0	7	2
Dynamo bogie including wheels, axleboxes, etc.	5	5	0	5	1	0		
57-foot underframe	7	6	2	6	14	0	12	2
Complete coach	30	6	0*	28	16	0	1½ tons	

* It should be noted that the previous standard coach of similar size and seating weighed 31 tons 11 cwts., but this has been reduced to 30 tons 6 cwts. to allow for the double bolsters in these coaches, thus giving a correct weight for comparison.

This saving in weight of 1½ tons as shown above is due to the application of welding, the elimination of floor framing, and other new design features which have been described previously. This coach was completed in April, 1934, and has been running since then on main-line trains with complete satisfaction.

Welded 12-Ton Open Merchandise Wagon.

In this country the standard open wagon is one of 12-ton capacity. These wagons are made either with steel or timber underframes. The steel underframes in the past have been built up of rolled sections riveted together, but this type of underframe will in time be replaced by electrically-welded underframes. The advantage of the welded underframe is that a reduction in weight of the wagon can be effected due to the elimination of rivet-holes, and the higher efficiency of the welded joint as compared with a riveted joint makes it possible to use lighter sections. Further, the elimination of angles, cleats and gussets necessary in riveted frames assists in the reduction of weight.

Many of the components required in wagon-construction, such as triangular crossbars, drawbar cradles, scroll irons, vee hangers, etc., can be fabricated by welding, thus effecting a saving not only in weight but also in cost.

The design and construction of the first 12-ton welded open merchandise wagon built by the L.M.S.R. in July, 1934, will now be described.

General Preliminary Considerations.

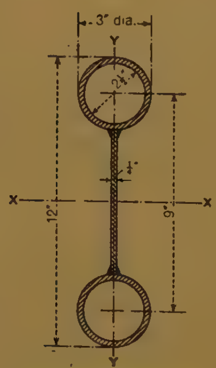
(*a*) *The efficiency of various steel sections for wagon-construction.* At the present time the channel section is largely used as a beam for wagon solebars, crossbars, and headstocks, but the question to be considered is whether weight for weight this is the most efficient section. In this connection a comparison of section moduli divided by areas in a number of cases is made in the following Table :—

Type of section	Channel	I Section	Tube	Bulb angle	Hollow elliptical	Hollow rectangle
Size of section	NBSC 10; 9-inch by 3-inch.	NBSB 7; 8-inch by 4-inch.	7·25 inches O.D. 6·75 inches I.D.	NBSBA 12; 9-inch by 3½ inch by 0·43-inch	Outside 10¼-inch by 4¼-inch. Inside 9¾-inch by 3¾-inch	Outside 8-inch by 3-inch. Inside 7·12 inch by 2·44-inch.
Weight per foot: lbs.	17·46	18·0	18·7	21·22	18·65	22·54
Sectional area: square inches	5·14	5·30	5·5	6·24	5·49	6·63
Section modulus (maximum)	13·89	13·91	9·28	13·92	10·53	13·63
Section modulus (minimum)	1·69	1·75	9·28	1·48	6·30	6·26
Modulus/area (maximum)	2·71	2·63	1·69	2·23	1·92	2·06
Modulus/area (minimum)	0·33	0·33	1·69	0·24	1·15	0·95

The following Table gives a comparison of the properties of a built-up tubular section (*Fig. 27*, p. 268) with channel sections suitable for a wagon solebar, and of a 6-inch by 3-inch angle with a 4-inch diameter tube for wagon-diagonals :—

Section.	Weight per linear foot: lbs.	Moment of inertia: inch⁴ units.		Modulus: inch³ units.	
		About XX	About YY	About XX	About YY
12-inch by 3-inch tubular, ¼-inch web	19·77	95·91	4·12	15·99	2·75
9-inch by 3 1/16-inch ; NBSC. 10	19·37	66·32	4·09	14·74	1·78
10-inch by 3-inch ; NBSC. 12	19·28	82·66	3·98	16·53	1·76
6-inch by 3-inch by ½-inch ; NBSUA	14·45	15·51	2·62	4·05	1·13
4-inch O.D. / 3¼-inch I.D. } tube	14·52	7·09	7·09	3·55	3·55

Fig. 27.



Scale: one-eighth full size.

Built-up Tubular Section.

A comparison of the figures in the above Tables indicates that from the point of view of resistance to lateral bending moments, the single tube or tubular built-up sections are more efficient than either the existing angle or channel sections. Also the built-up tubular section is stronger for the same weight than the existing channel for vertical bending moments. From the point of view of strut action, however, where the least radius of gyration or moment of inertia is the criterion, the channel appears to be as good as the built-up tubular section. The angle has not so great a resistance to end load as the simple tube of approximately the same area, but it has a greater resistance to vertical bending moments.

The question of using tubular sections or not depends, however, upon the cost of such sections compared with the ordinary rolled sections. The present cost of tubular sections is greater than that of the ordinary rolled sections, and as cost is one of the principal considerations in wagon-design, it was decided not to adopt tubular sections for the main frame members. Tubular sections, however, have been adopted for the triangular crossbars and spring-stops with resultant economy.

(*b*) *Buffing stresses.*—One of the most difficult aspects of wagon-design is the question of buffing stresses, as there appear to be no reliable data available on this subject except that it is known that the present steel frames withstand the average buffing shocks of traffic shunting. A force of 4·8 tons will close up an ordinary buffing spring (R.C.H. 1,018), but a rough shunt may double this, the buffing force becoming 19·2 tons for the whole wagon. On the other hand if it is assumed that a 12-ton standard merchandise wagon (tare 7 tons, load 12 tons), weighing 19 tons, is shunted at 20 miles per hour into a similar stationary wagon buffered up against an immovable block, the maximum force on the buffers would be about 47 tons weight; these assumed conditions are, however, very drastic.

Design of a Welded 12-ton Open Wagon.

A brief outline only of some aspects of the design will be given.

Loading.—Total weight = 19 tons
Weight of wheels, etc. = 2·5 „

$$\text{Loading} = 16{\cdot}5 \text{ tons.}$$

$$\text{Length of underframe} = 17{\cdot}5 \text{ feet.}$$

$$\text{Loading} = \frac{16{\cdot}5}{17{\cdot}5} = 0{\cdot}943 \text{ tons per foot run.}$$

The loading and bending-moment diagram is given in *Fig. 28* (p. 270).

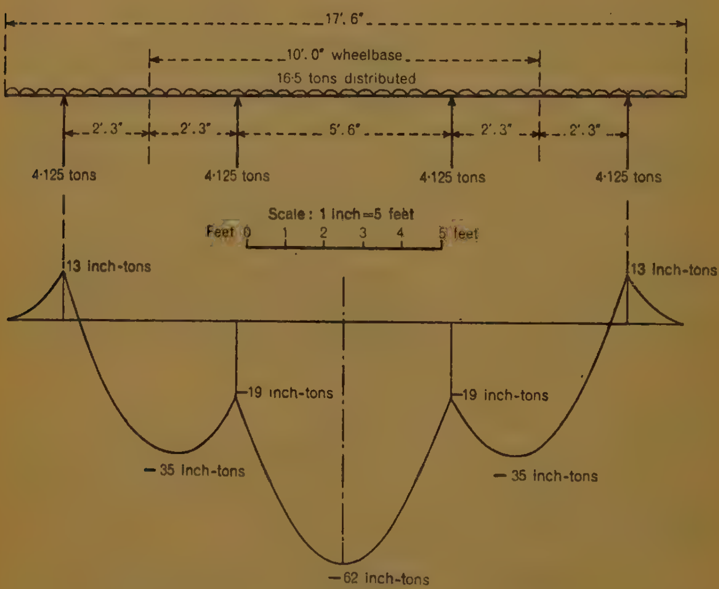
Solebar Section.—Here the question to be considered is whether the section can be reduced from standard on account of the elimination of rivet and other holes. The following holes have to remain :—

(1) $\frac{7}{16}$-inch-diameter holes in the top flange for floorboards.
(2) $\frac{13}{16}$-inch-diameter holes in the web for the brackets carrying the automatic-vacuum-brake cylinder.
(3) $\frac{13}{16}$-inch-diameter holes for the vee hanger in one solebar.
(4) $\frac{13}{16}$-inch-diameter holes for the axleguard riveting.

The weakest sections in the standard riveted solebar (*Figs. 29*) are :—

(*a*) At the centre of the solebar.
(*b*) 7 inches from the centre of the solebar.
(*c*) 1 foot $8\frac{1}{2}$ inches from the centre of the solebar.

Fig. 28.



LOADING AND BENDING-MOMENT DIAGRAMS FOR 12-TON OPEN WAGON.

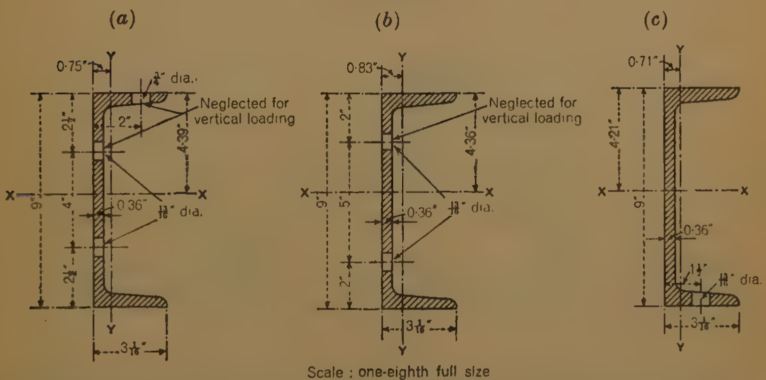
The moments of inertia in inch⁴ units for these reduced sections are :—

	Full section 9-inch by $3\frac{1}{16}$-inch channel.	Reduced section at (*a*).	Reduced section at (*b*).	Reduced section at (*c*).
I_{xx}	66·32	65·06	64·36	59·33
I_{yy}	4·09	3·37	3·86	3·86
Z_{xx}	14·74	14·12	13·86	12·38
Z_{yy}	1·78	1·61	1·73	1·64

In a welded design the solebar could be slightly reduced in section due to the elimination of rivet-holes if a satisfactory section could

be obtained. Using an 8-inch by $3\frac{1}{8}$-inch channel N.B.S.C. 8, with a thickened web of 0·405 inch, weighing 19·36 lbs. per foot, and having $\frac{13}{16}$-inch-diameter holes in the web, $I_{xx} = 50{\cdot}45$ inch^4 units, and $Z_{xx} = 12{\cdot}22$ inch^3 units. This section is weaker than the present solebar section. A 9-inch by 3-inch channel, N.B.S.C. 10, with $\frac{13}{16}$-inch-diameter holes in the web, giving $I_{xx} = 60{\cdot}75$ inch^4 units and $Z_{xx} = 13{\cdot}18$ inch^3 units, would have the required strength, resulting in a saving in weight of 33·4 lbs. per solebar. This section has to be rejected, however, on account of the web thickness being only 0·30 inch, which is not in accordance with railway practice. It is necessary that the solebar web thickness should be about $\frac{3}{8}$ inch

Fig. 29.



Sections at (*a*) Centre, (*b*) 7 Inches from the Centre, and (*c*) 1 Foot $8\frac{1}{2}$ Inches from the Centre, of a Standard Riveted Solebar.

on account of the reduction in the thickness of the web by corrosion during the life of a wagon, which is from 30 to 40 years. From the above considerations the Author decided to retain the 9-inch by $3\frac{1}{16}$-inch channel N.B.S.C. 10.

Buffer Trimmers.—The maximum load on the buffer spring was taken as 5 tons, as any greater load on the buffers will be taken on the headstocks owing to the construction of the buffer. The buffer load is distributed by the spring washer evenly over a 5-inch width, and the bending moments at various sections of the trimmer are calculated on the assumption of fixed ends. The loading and bending moment diagrams are given in *Fig. 30* (p. 273). Using $\frac{1}{4}$-inch fillets, the stress in the welding was calculated to be 3·53 tons per square inch.

Crossbars.—9-inch by $3\frac{1}{16}$-inch channels were retained for the crossbars, as from the calculated stresses a smaller section could

not be used. The position of the crossbars allowed a very simple type of block link-bracket to be used.

Block Link-Brackets.—These were fabricated from $2\frac{1}{4}$-inch diameter round bar, $2\frac{7}{8}$ inches long, welded to 2-inch $\times$ $\frac{7}{8}$-inch flat bar, $6\frac{7}{8}$ inches long, and welded to the crossbars (see *Figs. 31*). On test these block link-brackets withstood a tensile pull of 20 tons.

Scroll Irons.—The welded scroll iron used was of a very simple design (*Figs. 32*, p. 274). This scroll iron weighs 10·4 lbs. as against 16·5 lbs. for the standard cast-steel scroll iron which is riveted on to the solebar. The welded scroll iron was tested by applying a load, with the following results :—

Load : tons.	Deflection : inches.	Permanent set : inches.
1	0·02	Nil
3	0·04	,,
6	0·075	,,
9	0·11	0·025
12	0·19	0·09
15·4	Slight buckling of web of channel.	

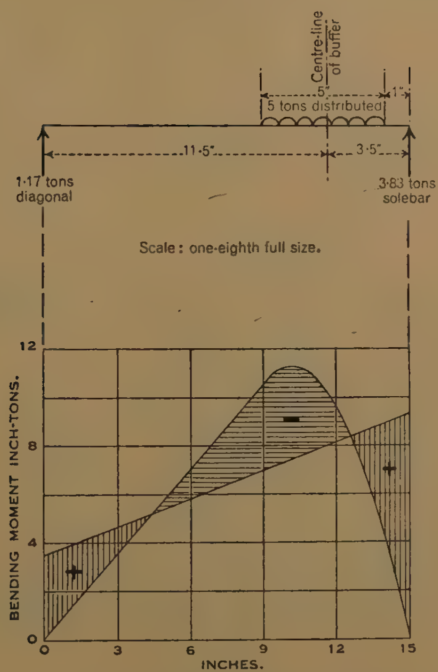
The standard cast-steel scroll iron was tested in a similar way but without being attached to a solebar, and gave the following results :—

Load : tons.	Deflection : inches.	Permanent set : inches.
1	0·015	Nil
3	0·03	0·01
6	0·06	0·01
9	0·09	0·02
12	0·12	0·03
20·5	Collapsed.	

It will be noticed from the latter test that a standard 12-ton wagon would carry a total of 160 tons on its scroll irons before failing, assuming, of course, that the solebar flanges would stand up to this load. From these tests it seems that these cast-steel scroll-irons are unnecessarily strong for the load which they have to carry.

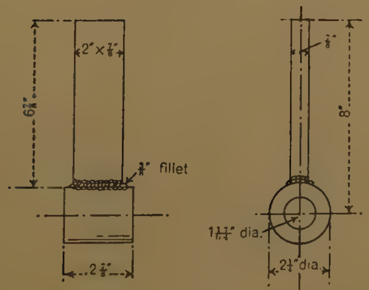
Axleguards.—Owing to the necessity for removing the axleguards for straightening when they have become bent due to shunting etc., it was not considered advisable to weld a standard axleguard on to the solebars. Consideration was given to the possibility of welding-

Fig. 30.



LOADING AND BENDING-MOMENT DIAGRAMS FOR BUFFER TRIMMER.

Fig. 31.

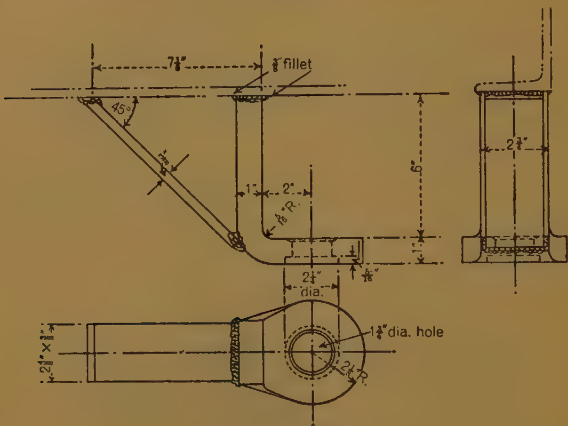


Scale: one-eighth full size.

BLOCK LINK-BRACKETS.

on a stiffened axleguard made of 9-inch by ¾-inch plate tapering to 3¾ inches by ¾ inch at the bottom. This stiffened axleguard and a

Fig. 32.

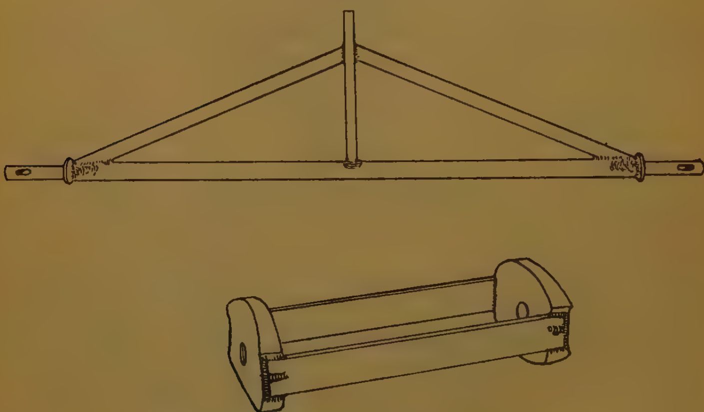


Scale: one-eighth full size.

WELDED SCROLL-IRON.

standard axleguard were tested by applying a load distributed over 4 inches at a distance 20 inches below the solebar. The standard

Figs. 33.



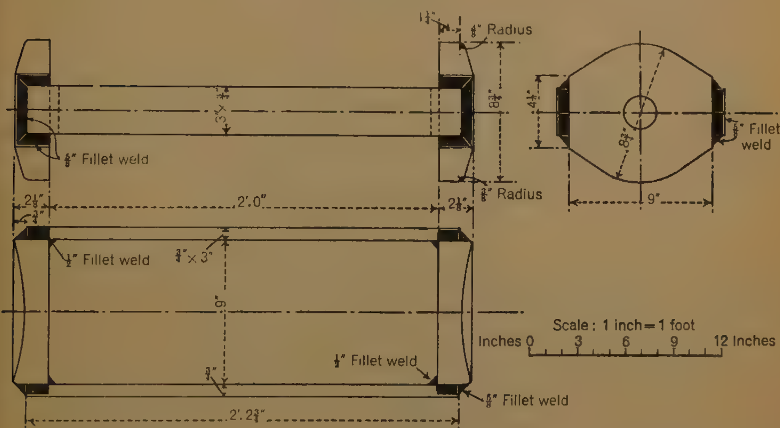
TRIANGULAR CROSSBAR AND DRAWBAR CRANE.

axleguard took 6·7 tons before the wing leg buckled on one side, whereas the stiffened axleguard took 10·7 tons, and failure took place by the solebar buckling. It will be seen from this test that

in the former case the failure could be put right by cutting the rivets and changing the axleguards, whereas in the latter case the solebar would have to be straightened, which is a major operation. Finally, a standard axleguard was used on the welded wagon and fixed on by riveting.

Drawbar Cradle.—The first design of welded drawbar cradle (*Figs. 33*) was tested to 100 tons tensile load before failure. A

Figs. 34.



simpler design of drawbar cradle (*Figs. 34*), having the 3-inch by $\frac{3}{4}$-inch side straps lapped on the ends for $1\frac{3}{8}$ inches and welded with $\frac{5}{8}$-inch fillet welds also withstood a 100-tons test without failure.

Vee Hanger.—The vee hanger was tested by applying a downward force through the bottom junction plate, and failure took place at 37·6 tons.

Triangular Crossbars.—The welded triangular crossbar is shown in *Figs. 33* and was made up from tubular sections, giving a reduction in weight of 36 lbs. for the long type.

Calculation of Weld-Fillet Sizes.—The same values for the strength of the weld metal, etc., as given on p. 240 were used for calculating the fillets for this wagon. The method for calculating fillets will be further illustrated by the following example, but it will be obvious that further considerations are necessary, although omitted in this case.

Diagonal and Crossbar Joint of Underframe.—The diagonal is a 6-inch by 3-inch by $\frac{1}{2}$-inch angle, and has to be flush on top with the 9-inch by $3\frac{1}{16}$-inch channel crossbar. Assuming that $\frac{5}{16}$-inch leg fillets

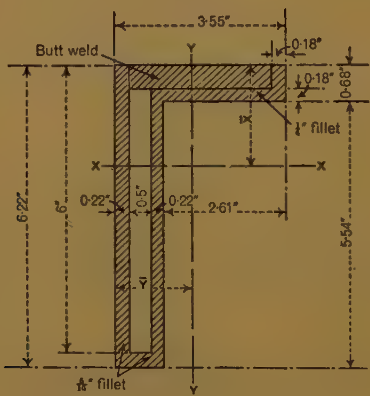
on each side of the vertical leg of the angle will give suitable strength, then combined with a ½-inch butt weld on the horizontal leg of the angle and with a ¼-inch weld on the underside, the joint will be as in *Fig. 35*, neglecting radii. The diagonal meets the crossbar at an angle, hence the length of top flange of the angle = 3 × sec (17 degrees 29 minutes) = 3·15 inches.

Then $\overline{X}$, the distance of the neutral axis from the top of the welding

$$= \frac{3 \cdot 55 \times 6 \cdot 22 \times 3 \cdot 11 - 2 \cdot 61 \times 5 \cdot 54 \times 3 \cdot 45 - 0 \cdot 5 \times 5 \cdot 5 \times 3 \cdot 25}{3 \cdot 55 \times 6 \cdot 22 - 2 \cdot 61 \times 5 \cdot 54 - 0 \cdot 5 \times 5 \cdot 5}$$

$= 2 \cdot 02$ inches.

Fig. 35.



Similarly, $\overline{Y} = 1 \cdot 15$ inches.

$$\text{Hence } I_{xx} = \frac{3 \cdot 55 \times 6 \cdot 22^3}{12} + 22 \cdot 08 \times 1 \cdot 09^2 - \left(\frac{2 \cdot 61 \times 5 \cdot 54^3}{12} + 14 \cdot 46 \times 1 \cdot 43^2\right) - \left(\frac{0 \cdot 5 \times 5 \cdot 5^3}{12} + 2 \cdot 75 \times 1 \cdot 23^2\right)$$

$= 19 \cdot 78$ inch^4 units,

and $$Z_{xx} = \frac{19 \cdot 78}{4 \cdot 20} = 4 \cdot 71 \text{ inch}^3 \text{ units.}$$

The corresponding I_{xx} and Z_{xx} for the parent metal of the 6-inch by 3-inch by ½-inch angle are 15·51 and 4·05, so that the welding should be satisfactory about XX.

Similarly,

$$\left.\begin{aligned} I_{yy} &= 4{\cdot}92 \text{ inch}^4 \text{ units} \\ Z_{yy} &= 2{\cdot}05 \text{ inch}^3 \text{ units} \end{aligned}\right\} \text{for the weld metal.}$$

$$\left.\begin{aligned} I_{yy} &= 2{\cdot}62 \text{ inch}^4 \text{ units} \\ Z_{yy} &= 1{\cdot}13 \text{ inch}^3 \text{ units} \end{aligned}\right\} \text{for the parent metal of the 6-inch by 3-inch by } \tfrac{1}{2}\text{-inch angle.}$$

Hence the proposed welding of this joint should be satisfactory, as Z for weld metal is more than 20 per cent. greater than Z for parent metal. These considerations show that the calculation of the stresses in welds, as explained on p. 252, will probably confirm the above provisional assumption of weld dimensions.

The final design for the 12-ton merchandise open wagon is given in Figs. 36, Plate 2, which also show the sizes of all the welds. It will be noticed that all welding dimensions given are in terms of the "leg" size.

Method of Construction.

The first step taken in connection with the construction of this welded wagon was to have all sections and component parts prepared to drawings, care being taken to see that all lengths, etc., were exact. Next the following components were fabricated :—

Block link-brackets.
Scroll irons.
Brake trimmers.
Vee hangers.
Triangular crossbars.
Drawbar cradle.
Eyebolt to door-hinge.
Buffer trimmers.

The following preliminary assembly was carried out in the welding shop :—

(1) Longitudinals : welding on drawbar-cradle carriers, one longitudinal having welded trunnion bracket.

(2) Crossbars : welding on block link-brackets.

(3) Headstocks : welding on drawbar plate and stiffener, coupling hook and two end stanchions.

(4) Side-rail angles : welding on door-hinge lugs, and doorway nosing-plate.

(5) Side stanchions : welding on diagonal bracing lugs.

(6) Solebars : welding on vee hanger, spring stops, brake-lever guard-brackets, scroll irons and gussets, plates for door bumper-springs, horse hooks, rope hooks, and side-rail brackets.

Having completed the fabrication of the components and the preliminary assembly, the frame was assembled on a table. The order followed in assembly and welding of the frame was as follows :—

(1) Longitudinals to crossbars.
(2) Crossbars to solebars.
(3) Diagonals to crossbars.
(4) Brake trimmers to diagonals.
(5) Buffer trimmers to diagonals and solebars.
(6) Diagonal stay to diagonal and headstock.
(7) Diagonals to headstock.
(8) Solebars to headstock.

All the welding was done in a horizontal position so as to avoid overhead and vertical welding, consequently the frame had to be turned into six positions by the crane.

D.C. welding and a high-class covered electrode having the following physical properties were used throughout :—

Tensile strength	= 32 tons per square inch.
Elongation on 4 diameters	= 26 per cent.
Elongation on 2 diameters	= 22 ,,
Izod value	= 69–76 foot-lbs.

The largest welds on the wagon underframe were $\frac{3}{8}$-inch leg fillets.

Details of welding similar to those described on p. 255 were recorded for the construction of this wagon.

Revolving jigs.

When a number of coach or wagon underframes are being constructed, it is an economical proposition to construct revolving jigs to facilitate the turning of the underframes into the necessary positions, enabling all the welding to be done "downhand," and so avoiding vertical and overhead welding.

Distortion and Shrinkage.—On making measurements of the relative heights of the four corners of the frame, the relative distortion was found to be $\frac{1}{16}$ inch only. The scroll-iron centres varied from 4 feet 6 inches to 4 feet $6\frac{3}{16}$ inches, the correct measurement being 4 feet 6 inches. The distance between solebars varied from $\frac{1}{8}$ inch to $\frac{3}{8}$ inch over the correct measurement.

Similar measurements were taken on a welded underframe for a 12-ton covered goods wagon, which was constructed later. It was found that the transverse shrinkage over the solebars varied from $\frac{1}{16}$ inch to $\frac{1}{8}$ inch, the longitudinal shrinkage over the headstocks varied up to $\frac{1}{8}$ inch, and the centres of the scroll irons were correct.

In this latter case the scroll irons were welded to the solebars after the underframe welding had been completed. These shrinkage figures are given for the first experimental underframes made without jigs, and now that jigs are being used, along with the experience gained, the shrinkage has been reduced to a minimum.

Miscellaneous.—After the welding was finished, the frame was fitted with riveted axleguards, and equipped with standard springs, axleboxes, wheels, brake-cylinder, timber body, etc.

Weight.—The complete wagon, fitted with vacuum brake, weighed 6 tons 9 cwts. 1 qr., being a reduction in weight of 6 cwts. 1 qr. as compared with a similar wagon with a riveted frame. The weight reduction on the frame alone amounted to about 20 per cent.

Testing.—The completed wagon was not submitted to any tests, as a similar welded frame had been tested to destruction with very satisfactory results, and, further, every stage of the design had been checked by practical tests. This welded wagon (*Fig. 37*, facing p. 263) was completed in July, 1934, and has since been running in traffic with complete satisfaction.

This work was carried out under the direction and supervision of Mr. W. A. Stanier, Chief Mechanical Engineer, and Mr. J. Purves, Assistant to the Chief Mechanical Engineer for Carriages and Wagons. The Author wishes to express his appreciation to Mr. Stanier and to the London, Midland and Scottish Railway Company for approval to give this Paper together with the photographs and drawings, and also to thank those members of the Chief Mechanical Engineer's Department who assisted with the work described in this Paper.

The Paper is accompanied by twenty-five sheets of drawings and nineteen photographs, from some of which Plates 1 and 2, the figures in the text and the four pages of half-tones have been prepared, and by the following Appendix.

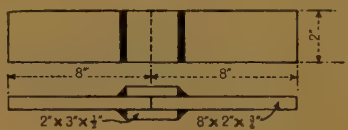
APPENDIX.

Examples of Electric Welding Tests.

1. *Joints subject to tension or shear.* *B.S.S. No.* 538.

P = Breaking load. L = Leg size of weld. f = Ultimate stress.

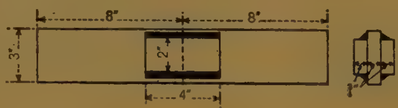
A.
End Fillet Welds.



Test (*a*).
P = 27·5 tons. $L = \frac{3}{8}$ in. $f = \dfrac{27{\cdot}5}{0{\cdot}265 \times 4} = 26$ tons per sq. in.

Test (*b*).
P = 28·8 tons. $L = \frac{3}{8}$ in. $f = \dfrac{28{\cdot}8}{0{\cdot}265 \times 4} = 27{\cdot}2$ tons per sq. in.

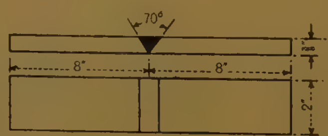
B.
Side Fillet Welds.



Test (*a*).
P = 36·5 tons. $L = \frac{3}{8}$ in. $f = \dfrac{36{\cdot}5}{0{\cdot}265 \times 8} = 17{\cdot}2$ tons per sq. in.

Test (*b*).
P = 32·7 tons. $L = \frac{5}{16}$ in. $f = \dfrac{32{\cdot}7}{0{\cdot}221 \times 8} = 18{\cdot}5$ tons per sq. in.

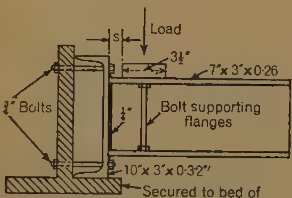
C.
Butt Welds.



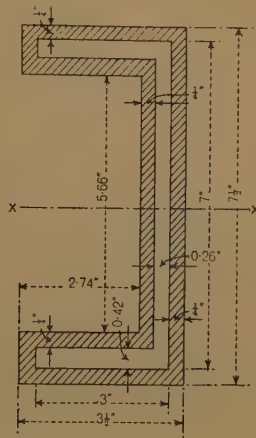
Test (*a*).
P = 21·5 tons. Throat $= \frac{3}{8}$ in. $f = \dfrac{21{\cdot}5}{0{\cdot}375 \times 2} = 28{\cdot}7$ tons per sq. in.

Test (*b*).
P = 21·65 tons. Throat $= \frac{3}{8}$ in. $f = \dfrac{21{\cdot}65}{0{\cdot}375 \times 2} = 28{\cdot}9$ tons per sq. in.

2. *Joints subject to bending moment and shear stress.*



METHOD OF TESTING JOINT.



$$I_{xx} \text{ of weld} = \tfrac{1}{12}(3{\cdot}5 \times 7{\cdot}5^3) - \tfrac{1}{12}(2{\cdot}74 \times 5{\cdot}66^3) - 32{\cdot}75$$
$$= 123{\cdot}2 - 41{\cdot}4 - 32{\cdot}75$$
$$= 49{\cdot}05 \text{ inch}^4 \text{ units} \quad \therefore Z = \frac{49{\cdot}05}{3{\cdot}75} = 13{\cdot}08.$$

Assume that for a $\frac{1}{4}$-inch throat weld the safe load = 11 tons.

(i) *When* $S = \frac{9}{16}$ *inch.*

$$f_1 = \frac{11}{2 \times 6{\cdot}16 \times 0{\cdot}25} = 3{\cdot}58$$
$$B.M. = 11 \times \left(\frac{3{\cdot}5}{2} + \frac{9}{16}\right) = 25{\cdot}44$$
$$f_2 = \frac{25{\cdot}44}{13{\cdot}08} = 1{\cdot}94$$
$$f = \sqrt{3{\cdot}58^2 + 1{\cdot}94^2}$$
$$= 4{\cdot}07 \text{ tons per sq. in.}$$

(ii) *When* $S = \frac{1}{4}$ *inch.*

$$f_1 = \frac{11}{2 \times 6{\cdot}16 \times 0{\cdot}25} = 3{\cdot}58$$

(see p. 252 for f_1, f_2, etc.)

$$B.M. = 11 \times \left(\frac{3{\cdot}5}{2} + \frac{1}{4}\right) = 22{\cdot}0$$
$$f_2 = \frac{22{\cdot}0}{13{\cdot}08} = 1{\cdot}68$$
$$f = \sqrt{3{\cdot}58^2 + 1{\cdot}68^2}$$
$$= 3{\cdot}96 \text{ tons per sq. in.}$$

Weld metal in side fillets should fail at not less than 18 tons per square inch (see B.S.S. 538 and tests 1 (B) above), the lesser value of stress in side fillets being taken in preference to the greater value of end fillets in order to be on the safe side. Hence the factor of safety should be not less than about 4·5.

Applying the factor of safety of 4·5 to the safe load of 11 tons gives 50 tons approximately as the ultimate load for the $\frac{1}{4}$-inch weld under the above conditions.

On test (i) failed at 58 tons and (ii) at 58·4 tons, hence for this particular case the method of calculation appears to be somewhat on the safe side.

At the conclusion of the Paper the Chairman of the Association, Mr. C. H. Bailey, proposed a vote of thanks to the Author.

The vote of thanks was carried by acclamation.

Discussion.

Mr. Carpmael.

Mr. Raymond Carpmael said that, on p. 233 the Author had stated that, as the result of a test, "about one hundred and six members were eliminated from the standard framing of the 57-foot corridor-third-brake coaches with a considerable saving in cost"; that was, presumably, the whole idea of the system of tests that was described in the Paper.

An idea of the completeness of the use of welding might be obtained from the Author's statement on p. 254 that "The only parts fixed by riveting or bolting are those which require fairly frequent removal, such as brake-work and axle-guard liners. Due to the elimination of rivet-holes, it has been found possible to use lighter sections . . ."; for many years it had seemed to him to be wrong, when fastening two pieces of steel together, to bore holes in them and then to fill up the holes with bolts, an extra thickness of metal being added to the plates to compensate for the weakening caused by the holes.

It was very interesting to learn about the extended use of welding in the L.M.S.R. wagon and carriage shops. Welding was used for some work on the G.W.R., such as for repairing coaches, but up to the present no large jobs, such as a roof or a bridge, had been constructed with welded joints; he hoped that such works would, in time, be welded. He would like to ask one question, namely, what tolerance was allowed on the fillet-welds, and what percentage of those fillet-welds outside the tolerance, or otherwise defective, would result in the rejection of a complete built-up frame?

Mr. Gibbins.

Mr. Frank Gibbins congratulated the Author on his Paper. In 1932 his firm, the Gloucester Carriage and Wagon Company, received an order for twenty-seven all-steel electric coaches for the South Indian Railway. The railway company were anxious to reduce the weight of their existing stock, and in collaboration with Mr. B. G. White, M. Inst. C.E., of Messrs. Robert White and Partners, the engineers for the railway company, it was decided that welding would be a big factor in achieving the required result. The designs were made accordingly. Each train was made up of three articulated coaches, and the whole of the bogies, underframes and bodies were welded with the exception of the body-panels. Every detail of the bogie was fabricated and welded, and all steel castings were eliminated;

the use of ordinary cast-steel brackets was eliminated, the brackets being made up of $\frac{3}{8}$-inch by $\frac{1}{2}$-inch plates. The underframe, which was 51 feet long, was made up of 6-inch channels, the two solebars running through in one length, whilst the central longitudes were in short lengths and welded to the crossbars. An interesting test was made with that underframe. It was slung up by chains 2 feet on either side of the centre-line, and the ends were then brought down 16 inches and released; the action of the frame was like a piece of spring steel, and on examination no set had taken place and no damage had been done to the frame. He would not like to test a riveted frame like that. With that coach 2 tons was saved by welding, or 6 tons per unit, which enabled an extra trailer car to be added to the existing power unit. Mr. Gibbins.

His firm had recently built some 65-foot 6-inch welded wagons, which he thought were the longest welded wagons running in Great Britain. The underframe comprised two fish-bellied members, top and bottom plates being welded to another plate to form a beam. The frame had an inverted camber of $1\frac{1}{8}$ inch between the bogie centres. A distributed test-load of 40 tons was applied to the wagons, and with that load the camber was reduced to $\frac{3}{8}$ inch; after 24 hours the load was removed and the camber returned to the original $1\frac{1}{8}$ inch. The weight saved by using welding on that wagon was 2 tons.

Mr. H. E. Allen remarked that he was especially interested in the Paper, as, in addition to having practised as a civil engineer, for some years he had been general manager of one of the large rolling-stock companies. At the time very little welding was employed except for small component members in the details of the underframe. With regard to welding, he would ask whether any microscopic examination had been made; if so, had any sign of failure been observed? Mr. Allen.

The Author stated that "All steelwork was prepared to correct length, etc., as shown in the drawings, except the solebars, which were left slightly longer." Solebars were usually difficult to handle, and they should be correctly milled to length. Were they left slightly longer to allow for camber? If the solebars had to be cut to length when in position, would it not add considerably to the cost?

Regarding the interior equipment of rolling stock, he would like to point out that the use of double sliding corridor doors made egress very awkward, while sometimes the doors jammed.

He had also travelled a fair amount on the Southern Railway's main-line electric services, and when running around curves the coach would start to swing, gradually increasing to a shaking action;

Mr. Allen. that motion was, he thought, due to the oscillation-springs being too weak. The travelling was very uncomfortable.

Professor Batho. Professor CYRIL BATHO remarked that he was surprised that all the tests described had been static tests, as he thought that a railway coach was continually subjected to vibration from braking and varying conditions of loading; further, he had always understood that weakness, when it existed in welded joints, was a weakness in fatigue-strength, so that he would have expected that dynamic tests would have been carried out, at any rate on the welded elements, and possibly on the whole frame.

Mr. Dunster. Mr. F. L. DUNSTER observed that, with the welded construction described, the body was built direct on to the underframe, whereas in the old method indiarubber body-pads were used; further, the new L.M.S.R. stock being built by various contractors, as well as that being constructed by the railway company, still employed body-pads. He would therefore ask the Author whether the welded cars described in the Paper were in the experimental stage, or whether the idea had been dropped and the old method been re-introduced.

Mr. Allen had commented on oscillation of the Southern Railway main-line electric trains; Mr. Dunster suggested that the oscillation might be caused from the fact that multiple-unit stock was employed, the train being driven both from the rear and from the front, giving rise to a pull-and-push action on the buffers. When ascending a gradient the leading motor-bogie would tend to sag relative to the rear bogie, thus making a certain amount of compression on the buffers, and creating a concertina action afterwards.

Mr. Crump. Mr. BENJAMIN CRUMP asked how the camber, which was put in the frame when setting out for welding, was retained. When cross pieces were welded on to the solebars distortion always resulted, and the length of the members always shortened. He would like to know what method was adopted for keeping the frame straight when the welding was done. What was the Author's objection to overhead welding and vertical welding? It was impossible to rotate a frame 57 feet long and to keep it perfectly in camber while the members were welded on. Further, the suppliers of electrodes had produced a rod which was perfectly good in vertical and overhead welding, as well as in the touch-rod or free-hand welding.

Mr. Cheesley. Mr. A. T. CHEESLEY remarked that he was very surprised that the L.M.S. Railway Company had spent so much money and time, and had given so much attention, to perpetuating the use of timber. Was it not time to adopt an all-steel construction? He thought that the results of the tests on the new design appeared to support that view, as the amount of steel used had been very considerably increased. Steel cant-rails, arch-rails and corner-rails had been

introduced, and he thought that coach-bodies should be constructed entirely of steel; at any rate, tests similar to those described should be carried out to compare the use of steel with that of timber. Mr. Cheesley.

He felt that the method of assembly of the frame left much to be desired, and he would mention, from his own experience, that there was a large amount of stress locked up in the frame which could be avoided; that opinion was apparently held by the Author, who stated, on pp. 257 and 258, that great care had been taken to prevent distortion and shrinkage, and that the total shrinkage of the carriage underframe was $\frac{5}{16}$ inch, while "other total shrinkages in width along the underframe varied from $\frac{1}{32}$ inch to $\frac{1}{8}$ inch." With regard to the wagon, the transverse shrinkage was from $\frac{1}{16}$ inch to $\frac{1}{8}$ inch, longitudinal shrinkage being up to $\frac{1}{8}$ inch. Mr. Cheesley noticed that the word "shrinkage" had been used, and he wondered if that meant that such a tolerance was necessary; he was sure that no inspector would pass a frame obtained from an outside supplier which departed from standard by anything like that amount. He did not consider that such tolerances as $\frac{3}{16}$ inch over the scroll-iron centres were necessary.

He observed that only arc-welding had been used. On the Continent, and in many other parts of the world, very little arc-welding was used in the construction of coaches and wagons, and he wondered why the L.M.S.R. had used arc-welding exclusively, instead of spot-welding. Remarkable Continental designs for rolling stock had been produced, but little was known of their costs. He would like the Author to give comparative costs of welded underframes for coaches and wagons as compared with the cost of riveted ones.

Mr. W. T. Tolhurst said that he was very pleased to note in the tests which the Author showed that on no occasion did the welding fail. The failure was always in the woodwork, and that being the case he thought that the L.M.S.R. had progressed so far that they should build all-steel carriages and wagons. Mr. Tolhurst.

With regard to the calculations of strength of the welds from the cross members to the solebars, the calculations for the bending moment, for instance, were, he thought, taken on the whole of the welding around the channel section. That hardly seemed right, because in the case of the bending moment the maximum stress was in the welds on the flanges; he would have thought that the safer way would have been to have calculated the moment of resistance of the section of the welds which connected to the flanges, since if a weld started to crack it would clearly start at those points. Although he was in favour of the use of welding, he was aware of its weaknesses. It was essential to remember them, especially when dealing

Mr. Tolhurst. with vibratory or alternating stresses. It was very important to base calculations only on the portions of the weld which were the most highly stressed. A very important case was where end standards were welded on to the headstocks. Only the top and bottom welds should be considered when calculating the strength, because they were the most highly stressed parts of the weld; if those welds failed, the side or vertical welds would obviously go very quickly. Recent tests had shown that weld metal, when subject to vibratory stresses, had failed at very much lower values than those found by the ordinary tensile tests. It was therefore essential, when dealing with joints subject to such stresses, to see that as much of the weld as possible was subject to a uniform maximum stress.

The last speaker mentioned that Continental wagons were largely spot-welded. He did not think it would be possible to spot-weld the underframes where fair-sized channel-sections were used, but he agreed that it would be advisable to attempt to spot-weld the panels on to the side frames when steel side frames were used.

Mr. Jackson. Mr. HARRY JACKSON remarked that there was one question he would like to ask regarding the order in which the welding took place in the production of the underframe. He thought that the Author had said that the welding was started at the middle and the welders worked towards the ends. His reason for asking the question was that, in the welding of some light lattice girders for a footbridge, special care had not been taken, and as a result considerable buckling had occurred in the main longitudinal member. Some such buckling might have taken place in the frames described by the Author, possibly resulting in unequal stress-distribution. Was that so?

Mr. Green. Mr. V. E. GREEN said that he would like to ask the Author whether any X-ray examination of the welding had been carried out during the early stages of construction. The point at which Mr. Green was most surprised was the general absence of diagonal bracing. Cruciform bracing was employed in modern motor-car construction, yet in the main underframe of the coach diagonal bracing was entirely lacking, although there was a certain amount of bracing in the vertical frames.

He thought that the new form of steel coach would introduce a fair amount of noise due to drumming. When travelling in G.W.R. trains he had been surprised at the drumming in the corridors of coaches with all-steel frames and steel panelling.

A point he wished to raise was the method of painting for protection from corrosion. When so much steel was introduced adjacent to the floor, he thought that corrosion might be experienced through dust from the track and from passing trains. From some tests he had carried out recently on wagons built about 50 years ago,

with wrought-iron axles, it was found that quite a number of axles had failed through corrosion fatigue; it seemed quite common, however, in lavatory coaches to see axles which were subject to corrosion fatigue from the ordinary discharge, and he thought that steps could be taken to obviate that occurrence. Mr. Green.

The AUTHOR, in reply, said that in determining the size of welds it was advisable to allow an extra $\frac{1}{4}$ inch or $\frac{1}{2}$ inch on the calculated length of the weld, as stated on p. 242. When inspecting welds, the throat dimension was tested with a weld gauge and no weld was passed which was under size. If the weld was below the specified size an extra run of weld metal was added to bring it up to size, and hence the possibility of having to reject a complete frame because the welds were under size did not arise in practice. The Author.

He was interested in the details given by Mr. Gibbins of all-steel welded coaches for the South Indian Railway. Mr. Gibbins mentioned that the order for those coaches was received by the Gloucester Carriage and Wagon Company in 1932, and in *The Welder* of February, 1934, was a description of these coaches, with the remark that the Gloucester Carriage and Wagon Company "have just completed" the order. It was interesting to note from that description that the saving in weight on the welded underframes was nearly two-thirds of a ton, or about 13 cwts., which weight reduction was about equal to that obtained by the Author.

With regard to the 65-foot 6-inch welded wagons for transporting rails and ballast, according to *The Welder* of December, 1935, those bogie wagons weighed 18 tons 2 cwt. It would be found that the 2 tons reduction in weight represented about the same percentage reduction in weight on the welded underframe as was obtained on the wagon described by the Author. In the latter case the weight reduction on the frame alone was about 20 per cent.

Mr. Allen had asked whether any microscopic examination had been made of the welds. No microscopic or X-ray examination had been made of the finished welds as no really satisfactory method of doing so had yet been devised. Many microscopic tests of the weld metal had been carried out in the Company's laboratories. As regards the solebars being left slightly longer, that was done on the first coach underframe as an experiment to ensure that any shrinkage due to welding would be allowed for. The ends were then carefully finished to size, but on later underframes the ends were milled as the necessary length had then been determined. The matter of double sliding doors was irrelevant to the Paper, as they were not used in the design described nor had they been referred to. Similarly, the riding of the Southern Railway's main-line electric coaches was not a matter which came within the scope of the Paper.

The Author. Professor Batho had mentioned that the tests were static and not dynamic. Admittedly, some of the tests were static, but dynamic tests were carried out also, such as the alternating tests referred to on p. 257, impact tests on welds, and shunting tests on the coach underframe and bogies. A welded wagon was given shunting and impact tests at speeds ranging from 5 up to 30 miles per hour, and was finally tested to destruction to determine the value of the welds under dynamic conditions. Further, the Author made a thorough search of literature on welding published throughout the world, and took full cognizance of welding tests and the relation of dynamic, fatigue, impact and static values, before determining the values of the permissible stresses given on p. 240 which were fixed for the dynamic conditions of railway vehicles. Probably few coaches or wagons had been the subject of so great a number of tests of every kind, and the completely successful running of the vehicles described over a period of 2 years vindicated the methods adopted.

In reply to Mr. Dunster, the Author had stated that no rubber pads were placed between the steel key-sheeted floor and the underframe. Experiments were made to determine whether that type of floor allowed more noise to enter the coach than in the case of a coach having rubber pads, but no difference could be determined. It might be added that it was a well-established practice on several railways to put a steel floor directly on to the underframe.

Mr. Crump had asked how the camber was retained in the underframe, but no difficulty whatsoever was experienced in that direction. In the Author's opinion it was an exaggeration to state that "it was impossible to rotate a frame 57 feet long and to keep it perfectly in camber while the members were welded on." If proper attention were paid to welding procedure no difficulty would be experienced in keeping the camber in the underframe, but it should be borne in mind that the design had to be one that was suitable for welding, otherwise some difficulty might be experienced.

With regard to vertical and overhead welding, it was best avoided, as the possibility of a poor-quality weld was much greater than with downhand welding, as practical experience of welding would quickly prove. The extra cost of electrodes suitable for overhead and vertical welding would more than pay for the cost of rotating jigs if a number of underframes were being built. Several speakers had expressed their opinions regarding all-steel coaches and wagons, but that question of the L.M.S.R. Company's policy was not within the scope of the Paper.

The Author was very surprised at Mr. Cheesley's remarks with regard to the method of assembly of the coach underframe, and he suggested that such remarks were not based on practical welding

experience; the Author was fully satisfied with the methods he adopted, as the locked-up stresses were reduced to a minimum by the control of welding procedure and the use of electrodes of high ductility. With regard to the fact that "Mr. Cheesley noticed that the word shrinkage had been used, and he wondered if that meant that such a tolerance was necessary," the Author had stated that it had to be borne in mind that shrinkage was one of the principal difficulties experienced in welding, and that it was practically impossible to eliminate it; by experience, however, it was possible to reduce it to a minimum and to produce welded frames as accurate as riveted frames were. Tolerance was a totally different matter from shrinkage, and no welding tolerance was stated in the Paper, but if a tolerance were fixed the degree of accuracy that the work warranted would have to be considered, and a long time would elapse before wagon frames would be built to micrometer gauges. The Author.

Spot-welding was a useful process for joining thin plates, but it had no advantage over electric-arc welding for building underframes. With electric-arc welding it was possible to obtain up to a 100-per-cent. joint efficiency, whereas with spot-welding the maximum efficiency would be no higher, in most cases, than that of a riveted joint. The Author had tested a number of spot-welded joints similar to those that would be used for wagon frames, and he was not very satisfied with their quality, although they had been supplied by one of the firms dealing with that class of welding plant.

The Author was interested in Mr. Tolhurst's remarks regarding the calculations of welds. It was very important in calculating the dimensions of welds to take into consideration not only the bending moments referred to by Mr. Tolhurst, but also shear and other stresses. It would be seen, on p. 252, that the stresses in the flange welds due to bending moments had been included in the calculations. The Author was of the opinion that the subject of the design of welds had not received the investigation and research that such an important part of the development of welding technique warranted.

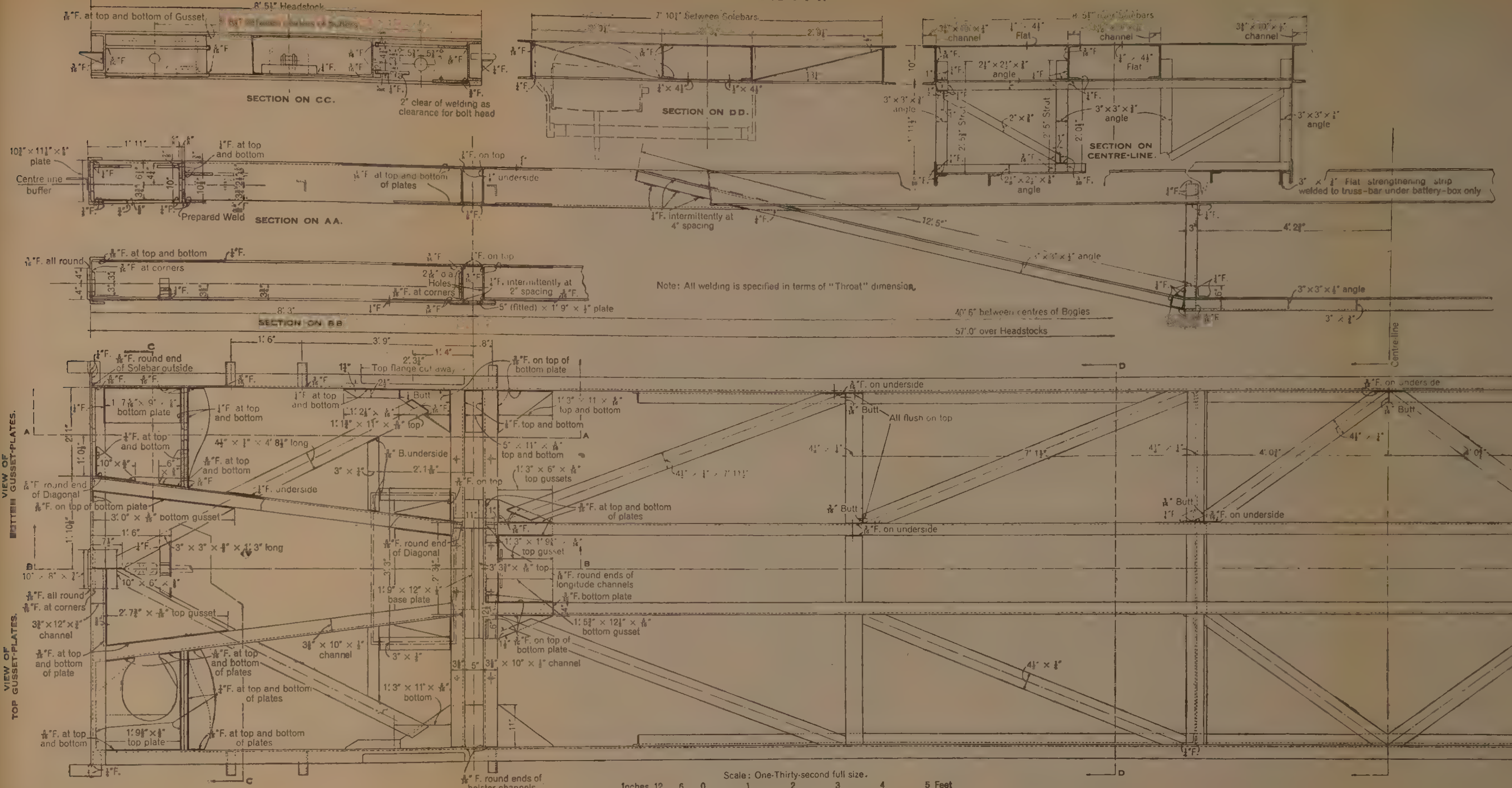
With regard to the question by Mr. Jackson as to the order of welding, it would be a little difficult to state methods which would be applicable to all classes of fabrications, as it was largely a matter of experience. The methods mentioned on pp. 254–6 and 277–8 had produced very satisfactory results. Mr. Green had referred to the absence of diagonal bracing, and stated that "in the main underframe of the coach diagonal bracing was entirely lacking." That was an ill-considered statement, and he would suggest that if Mr. Green examined the drawing of the coach underframe he would find it adequately braced. Experiments had been made to determine if the new type of coach was noisier in running than other types of

The Author. coaches, but practically no difference could be ascertained. The underframes were painted with a specially-prepared paint and the key-sheeting used for the floor was galvanized, and so far no trouble had been experienced due to corrosion from moisture.

The Council invite written communications on the foregoing Paper, which should be submitted not later than 2 months after the date of publication. Provided that there is a satisfactory response to this invitation, it is proposed in due course to consider the question of publishing such communications, together with the Author's reply.

SOME DEVELOPMENTS IN RAILWAY CARRIAGE AND WAGON CONSTRUCTION.

FIGS. 14.



ARRANGEMENT OF WELDED CARRIAGE UNDERFRAME.

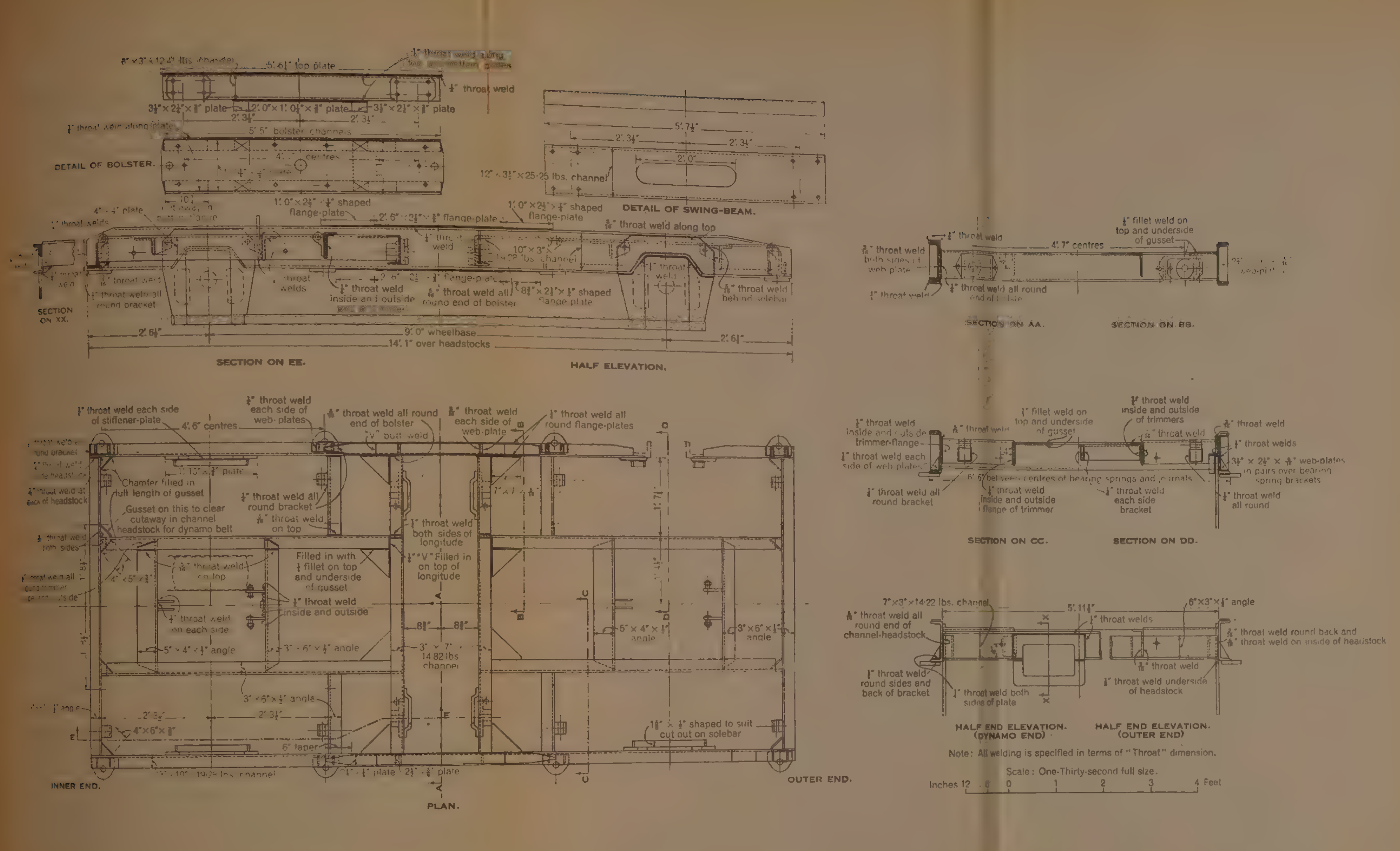
The Institution of Civil Engineers. Journal. June, 1936.

P. L. HENDERSON.

WATERLOW & SONS LIMITED, LATE THOS. KELL & SON, LONDON.

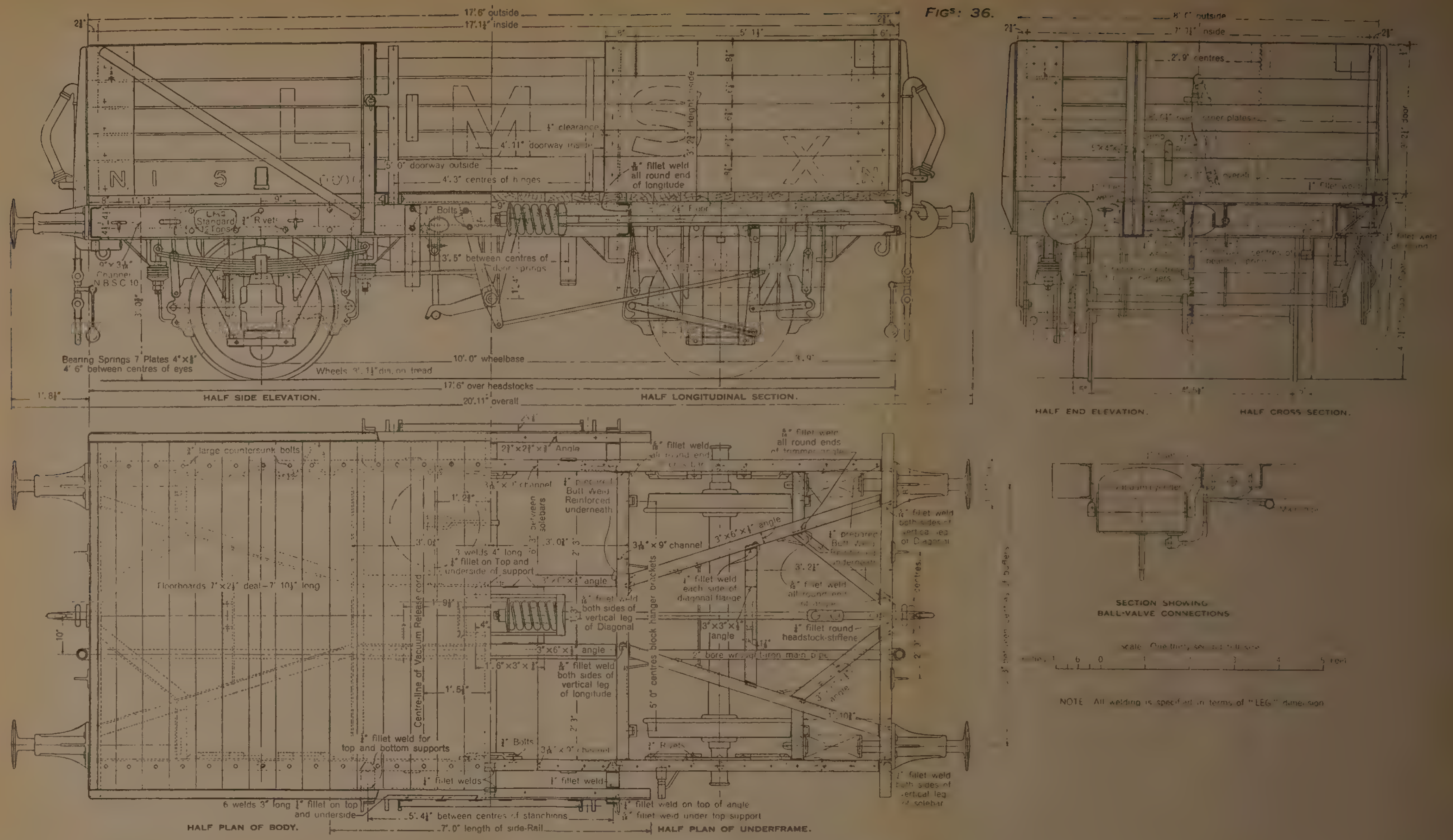
SOME DEVELOPMENTS IN RAILWAY CARRIAGE AND WAGON CONSTRUCTION.

FIGS: 18.



ARRANGEMENT OF WELDED CARRIAGE BOGIE.

FIGS: 36.



GENERAL ARRANGEMENT OF WELDED WAGON.

P. L. HENDERSON.

Paper No. 5061.

"The Redistribution of Moments in Reinforced-Concrete Beams and Frames."

By William Henry Glanville, D.Sc., Ph.D., M. Inst. C.E., and Frederick George Thomas, B.Sc., Assoc. M. Inst. C.E.

(Ordered by the Council to be published without oral discussion.)

TABLE OF CONTENTS.

Introduction.

It has long been known that concrete maintained under load within the working range will continue to deform for considerable periods and that the deformation may amount to four or five times the elastic movement. Evidence has been produced also to show that at high stresses, prior to failure, additional deformations of considerable magnitude may be expected.[1] Steel, however, exhibits no

[1] W. H. Glanville, "Creep of Concrete under Load." Proc. Inst. Struct. Eng., vol. 11, pt. 2 (1933), pp. 54–68.

appreciable progressive deformation under working loads, but at the yield point (or before failure, with steels having no well-defined yield point) it deforms by an amount several times the elastic movement.

As a direct result of these large inelastic deformations, redistribution of bending moments would be expected in most statically-indeterminate systems before final failure takes place. The term " redistribution of bending moments " is used to describe a departure from the distribution that would occur in a purely elastic framework. The deformations will in effect reduce the moments of inertia of the sections where they occur, causing a change in the moment-distribution throughout the system. In the case of steel structures the yield at joints is well known, no further moment being taken at a joint as a result of increased load after yield has commenced. It seemed reasonable to assume in a similar way for reinforced concrete that, should failure be approached at any section, the inelastic deformations of steel or concrete would affect the moment-distribution in such a way that further load would cause no change in the moment at the affected part.

The ordinary reinforced-concrete building is really a very highly-indeterminate framework, and the exact analysis of the moments and stresses using the ordinary elastic theory is more often than not extremely complicated, if not impossible. In order to simplify the design procedure the reinforced-concrete designer, although fully aware of the monolithic nature of the structure, has, therefore, been forced to ignore certain of the redundant features. For example, it has been the general practice to consider the beams and slabs as continuous and to consider the columns as acting separately, assuming that the column loads are axially applied. Experience has shown that the disregard of the bending moments in the columns has resulted in safe buildings except in extreme cases. The Code of Practice for the Use of Reinforced Concrete in Buildings [1] recommended revised methods of column-design using higher stresses than formerly, and it was not quite certain that with these revisions it would be safe to ignore bending in columns to the same extent as hitherto. Provision was therefore included for the approximate calculation for bending in those columns where it is likely to be most pronounced, namely, external columns.

The investigation described in the present Paper was undertaken in co-operation with the Reinforced Concrete Association to obtain definite information on this point in order to reduce, if possible, the

[1] " Report of the Reinforced Concrete Structures Committee of the Building Research Board, with Recommendations for a Code of Practice for the Use of Reinforced Concrete in Buildings." H.M. Stationery Office. 1933.

work of the designer and to avoid wastage of material in making columns unnecessarily strong. At the same time it was intended that information should be obtained on the allied problems of the amount of tension and compression steel that it was advisable to provide over supports and at column-junctions in continuous beams.

In order to illustrate the process of redistribution, the simple case of a built-in beam carrying a central load in the span may be considered. According to the elastic theory, on the assumption that the moment of inertia is constant throughout the length of the beam, and neglecting the weight of the beam, the sections should be designed to resist the same moments at the mid-span and the supports. If, however, the sections at the supports are weaker than at mid-span, incipient failure will be reached at the supports whilst there is still a reserve of strength in the span. In general, failure of the beam will not then occur, since large inelastic movements take place at the support without increase in moment, whilst the bending in the span is increased. It follows, therefore, that for every increase of load after the maximum strength of the support section has been reached, the whole resistance is supplied by the span-moments until a point is reached at which the span also fails. This, of course, assumes that the limiting deformation that can occur at the support is not exceeded before general failure.

Subject to this provision for the present case, therefore, the actual values of the moments that the separate sections can take are not important from the point of view of the ultimate strength of the beam, as long as the sum of the moments that the sections at midspan and support are capable of carrying is equal to the "free" bending moment, and provided that intermediate sections have been designed accordingly.

The Paper is divided into two parts, the first dealing with built-in and continuous beams and the second with frames.

PART I.

Reinforced Concrete Built-in and Continuous Beams.

The tests described in the first part of this Paper have been carried out to determine the extent of the redistribution of moments that may occur in reinforced-concrete beams, both at working loads and near failure, as a result of inelastic deformation of either the concrete or the steel.

The results of some tests to destruction on two-span continuous

beams have been published by G. von Kazinczy,[1] who showed that, when the steel is the deciding factor for failure, variation of the amount of steel in the span or over the central support from that required by the elastic theory leads to redistribution of moments such that the full strengths of both the span and support sections are reached. Similar results were obtained for built-in beams by the German Reinforced Concrete Committee [2] for the condition of failure due to steel-yield. Such redistribution is to be expected because of the large inelastic deformation of steel at its yield, but the extent to which it can be relied upon without causing concrete-failure is unknown. No previous tests are known in which the redistribution of moments has been observed as the result of the inelastic deformation of the concrete before failure.

The present tests, carried out at the Building Research Station, comprised :—

A. Tests at working loads on built-in beams.

B. Tests to destruction on two-span continuous beams designed as follows :—

(1) With weakness over the central support due to the use of a low percentage of tension steel.
(2) With weakness over the central support due to the use of a low-strength concrete without compression reinforcement.
(3) As (2), except that compression reinforcement was provided.
(4) As (2), but with an increased span-length in order to reduce shear stresses.
(5) As (2), except that a low-strength concrete was used at an age of about 6 months instead of 7 days.

All tests were made in duplicate, and river aggregates were used throughout.

Tests at Working Loads on Built-in Beams.

Calculations making allowance for creep under load had indicated that only a small readjustment of moments would occur as a result of creep of the concrete at working stresses. As a check, two small frames (beams 1 and 2) were tested to determine the effect in a built-in beam. The reinforcement of the frames and the arrangements for testing are shown in *Figs. 1* and *2*. In the first test the columns were cast vertically 24 hours before casting the beam ; in the second test the whole frame was cast on its side at one mixing. Particulars of

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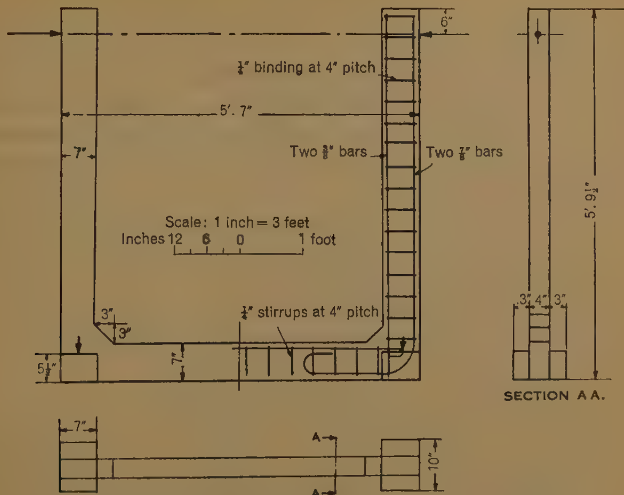
[1] G. von Kazinczy, "The Plasticity of Reinforced Concrete." *Beton u. Eisen*, vol. 32, pt. 5 (1933), pp. 74–80.

[2] C. Bach and O. Graf, "Versuche mit eingespannten Eisenbetonbalken," *Deutscher Ausschuss für Eisenbeton.* Heft 45. 1920.

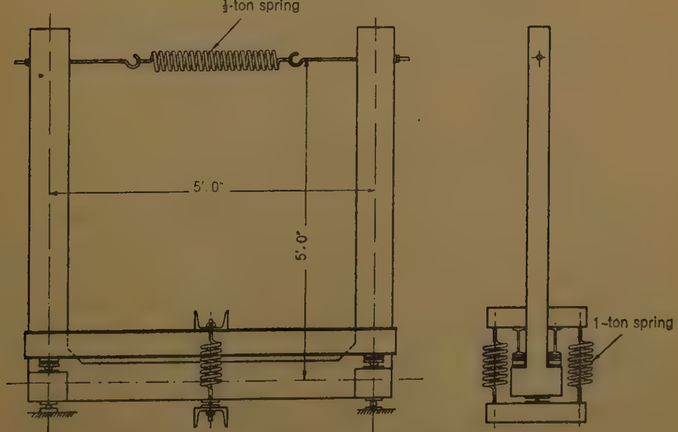
Fig. 1.



TEST ON BUILT-IN BEAM.

the concrete are given in Appendix I (p. 328). For convenience the frame was tested in an inverted position. At an age of 12 to 13 days the beams were loaded at midspan by means of two helical springs, the reaction being taken up through steel joists by special projections at the beam—column junctions. A third spring placed between the free ends of the columns was used to apply a bending moment at the beam ends, and was adjusted, as necessary, to give complete fixity at

Fig. 2.

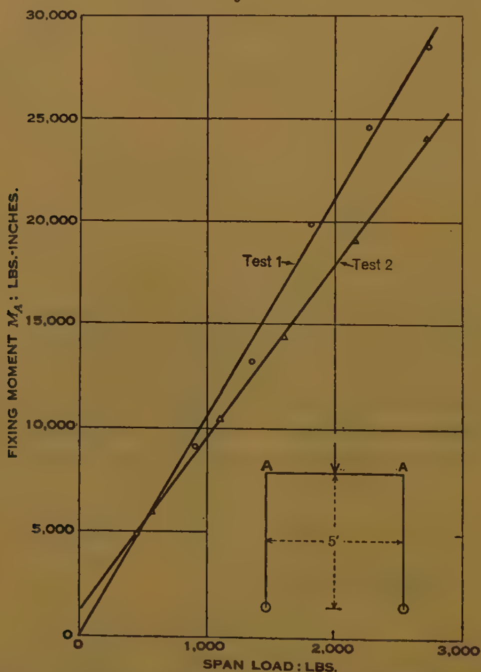


LOADING ARRANGEMENT FOR BUILT-IN BEAMS.

these points. This condition was realized by observing the rotation of mirrors mounted on rods cast in the ends of the beam and altering the tension of the third spring and, therefore, the fixing moment until the relative slope change between the ends of the beam was zero. The end moments required for fixity during loading are shown in *Fig. 3*, which indicates the reliability of the method used.

After the load had been brought to the design value (using stresses of 18,000 lbs. per square inch for the steel and 750 lbs. per square inch for the concrete) the load was maintained constant. The beam

Fig. 3.



TESTS ON BUILT-IN BEAMS; MOMENTS DURING LOADING.

slopes were periodically adjusted by altering the end moments to retain the condition of fixity. The results for the prolonged loading period are shown in *Fig. 4*, from which it will be seen that there was little change in the end moments, thus confirming the calculated results. Tests carried out in America [1] have yielded similar results.

After the period of prolonged loading, one of the frames was tested

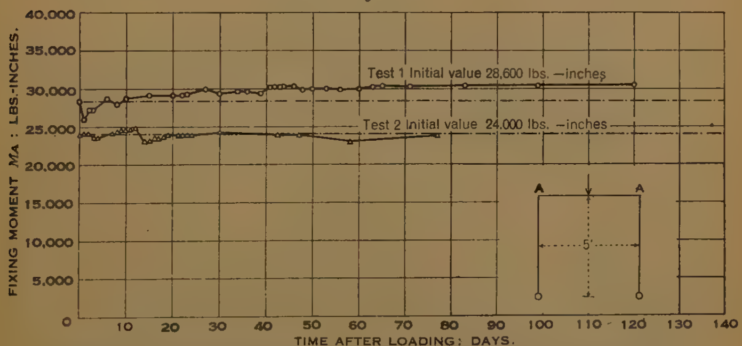
[1] F. E. Richart, R. L. Brown and T. G. Taylor, "The Effect of Plastic Flow in Rigid Frames of Reinforced Concrete," Journal Am. Concr. Inst., vol. 5, pt. 3 (1934), pp. 181–95.

to destruction as a pin-jointed portal with the feet joined by a steel rod the extension of which enabled the fixing moments to be determined. In this test there was definite evidence of redistribution of moment at incipient failure at the ends of the beam. The higher load carried by the frame as a result of redistribution caused shear failure in the beam, thus emphasizing the importance of proper design of shear reinforcement where redistribution of moments is taken into account.

Tests to Destruction on Two-span Continuous Beams.

(1) *Primary Failure in Tension Steel.*—Details of the beams and the positions of the loading used in the tests to determine the effect of using insufficient steel when calculated according to the ordinary

Fig. 4.



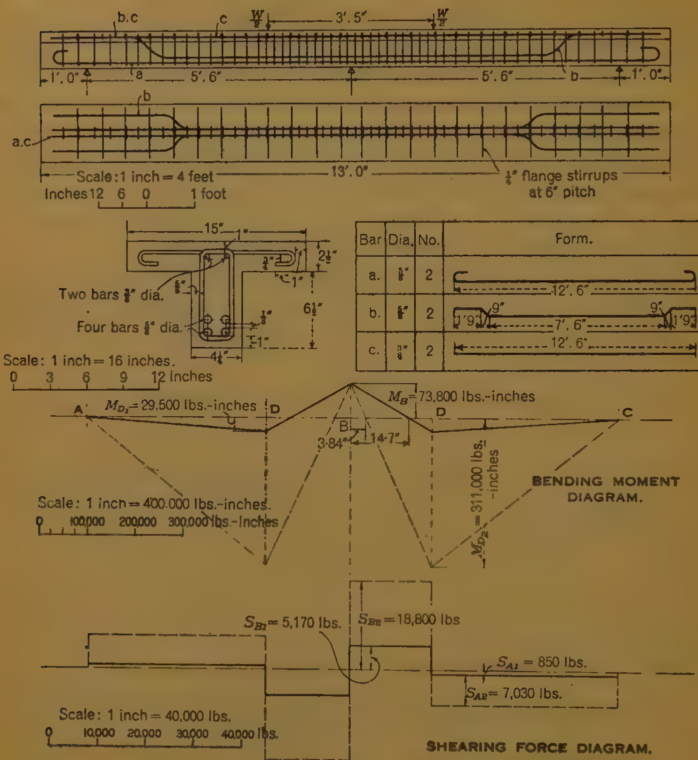
TESTS ON BUILT-IN BEAMS; MOMENTS UNDER CONSTANT LOAD.

elastic theory are given in *Fig. 5* (p. 298). The notation used in the Table of *Fig. 5* and in subsequent Tables of stresses is as follows :—

t denotes the stress in longitudinal tension reinforcement,
c ,, concrete stress,
M ,, bending moment,
n ,, depth of neutral axis,
a ,, arm of resistance moment,
S ,, total shear,
s ,, shear stress,
t_w ,, stress in web reinforcement,
s_b ,, bond stress,
R_A, R_B, R_C ,, reactions at A, B, and C.

It will be seen that over the central support, where the moment is normally greatest, there are only two $\frac{3}{8}$-inch diameter bars, whereas in the span four $\frac{3}{8}$-inch diameter bars are provided. At quite a low load, therefore, the yield-point stress of the $\frac{3}{8}$-inch diameter bars

Fig. 5.



Calculated Stresses. (*Lb.-inch units; notation as given on p.* 297).

Stage.	Support. At B. t	c	M	n	a	S	s	t_w	s_b	Span. At D. t	c	M	n	a	s_b	At A. S	s	t_w	s_b	Load W.	Dist. of point of inflexion from B.	Re-actions. R_B	$R_A = R_C$
1	44,700	2,545	73,800	1·85	7·47	5,170	162	6,255	293	3,740	285	29,500	2·09	6·40	103	852	28	2,170	30	10,980	14·7	10,340	852
2			73,800			18,800	592	22,700	1,073	39,400	3,000	311,000			374	7,030	231	17,800	251	50,750	3·94	37,600	7,030

Dead-load effects.

M_B = 3,800 lbs.-inches. R_A = 204 lbs.
M_D = 0 lbs.-inches. R_B = 476 lbs.

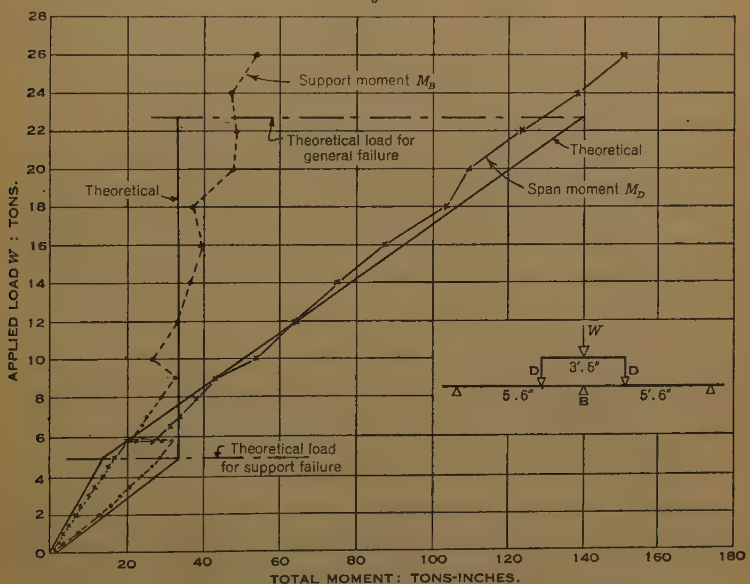
Average u = 8,400 lbs. per square inch. m = 5·5.

CONTINUOUS BEAMS RM2 (a) AND RM2 (b): STEEL FAILURE.

would be expected ; it would be anticipated that yield of these bars would lead to a redistribution of moments whereby the section over the central support would be continuously relieved of any additional moment, enabling the load carried by the system to be further increased until failure in the span.

The actual moments during the tests were determined by measuring the strain in the supporting steel joist at a fixed distance from the end supports and hence calculating the end reactions from a previous calibration of the joist. The results for the two beams tested are shown in *Figs. 6* and *7*. Incipient failure over the central support

Fig. 6.



TESTS ON CONTINUOUS BEAM RM2 (a) ; STEEL FAILURE.

is clearly indicated by a sudden decrease in moment at that point, after which the moment increased to a certain extent.

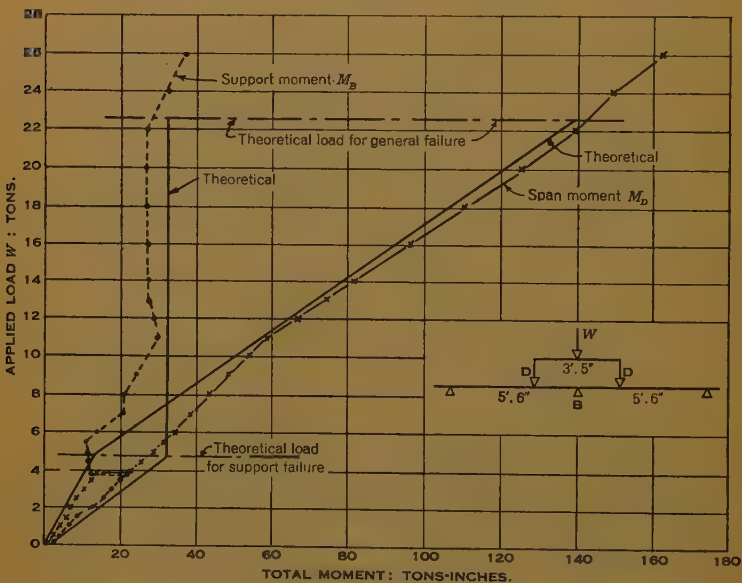
On the assumption that the central-support moment remains constant after yield begins, the subsequent moments in the span have been calculated and the theoretical curves are shown on the diagrams. It is evident that the assumption leads to a very fair estimate of the actual span-moments for the present tests.

The concrete used for the first beam tested (RM2 (a)) was made in the proportions 1 : 2 : 4 (by weight), using high-alumina cement, and the beam was tested at an age of 6 days. For the second beam (RM2 (b)) a rapid-hardening Portland cement 1 : 1 : 2 concrete (by

weight) was used, and the beam was tested at an age of 44 days. The higher tensile strength of the high-alumina cement concrete (see Appendix I, p. 328) accounts for the higher load at which the yield of the steel was reached for beam RM2 (a). With this beam, the help afforded by the concrete in tension was such that the stress in the continuity steel increased from a very low value to its yield-point value at the occurrence of the first crack over the support. Apart from this effect, there is apparently no important difference in behaviour resulting from the use of the two types of cement.

The deflections at midspan relative to the central support were

Fig. 7.



TESTS ON CONTINUOUS BEAM RM2 (b); STEEL FAILURE.

measured throughout by means of dial gauges. There was no appreciable difference between the deflections of the two beams, and at three-quarters of the failing load the maximum deflection was only about 0·1 inch. The supporting steel joist deflected during the test, and the sinking of the end supports relative to the central support was therefore also measured. This sinking affected the moments during the elastic stage of the test, and has been taken into account in calculating the theoretical curves and stresses given in *Figs. 5, 6* and *7*.

The maximum crack-widths, measured with a portable microscope, are given in Table I (p. 324). The cracking over the central support

increased considerably during the second part of the test, that is, after the steel had commenced to yield, and shortly before final failure the cracks were from 0·06 to 0·08 inch wide. These cracks are approximately ten times the width usually observed just before the commencement of steel-yield.

The loads calculated for failure, (i) on the elastic theory, and (ii) on the basis that both the support and span sections develop their full strengths after redistribution, are given in Table II (p. 325), together with the actual failing loads. It will be seen that the effect of the redistribution of moments on the load-carrying capacity of a continuous beam may be considerable in cases of steel-weakness over the central support. However, the cracking accompanying the increased load is very marked, so that, in practice, advantage can be taken of moment redistribution due to steel-yield only in cases where the increased cracking is not a matter of importance.

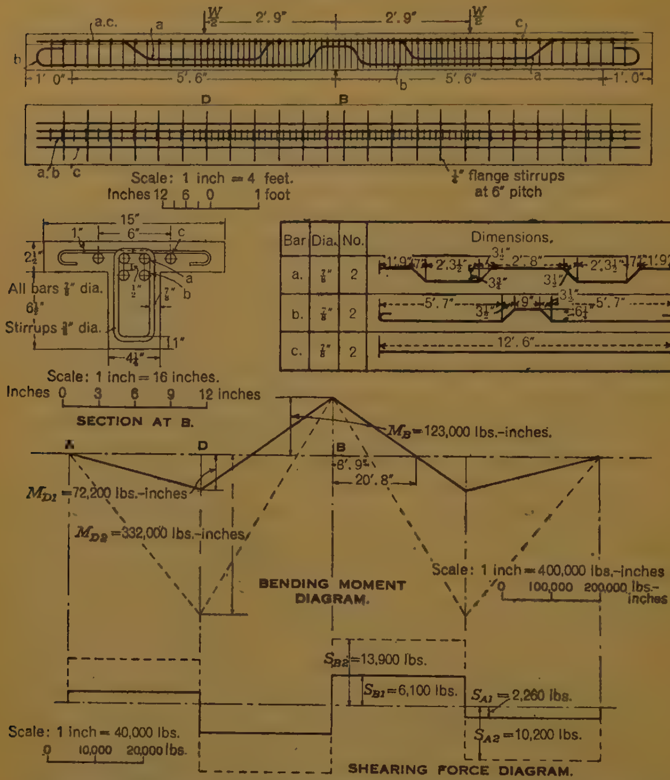
(2) *Primary Concrete Failure: No Compression Steel Provided over Central Support.*—In the beams designed to fail by crushing of the concrete, all the tension reinforcement in the span was taken up over the support so that the compression at that point was taken wholly by the concrete in the rib. Details of the beams, spans and positions of loads are given in *Fig. 8* (p. 302), the notation employed being as given on p. 297. The concrete was made with ordinary Portland cement using a 1 : $2\frac{1}{2}$: $3\frac{1}{2}$ mix (by weight) and a water—cement ratio of 0·66 (by weight). The tests were made at an age of 7 days, the strength aimed at being the lowest (2,250 lbs. per square inch) allowed by the Reinforced Concrete Code of Practice. Actually the strength was about 10 per cent. less than this value (see Appendix I, p. 328). In order to reduce the shear stresses with this weak concrete, the loads were applied at midspan, instead of nearer to the central support as in the case of the previous beams.

The results are given in *Fig. 9* (p. 303). It will be noticed that there is not such a well-defined point at which failure over the support commences as in the case of the previous beams in which the steel yielded, but rather a gradual change from the elastic to the inelastic stages of the test.

The concrete at the support continued to carry load in an apparently undistressed condition long after the load calculated to produce a fibre stress equal to the cube strength had been reached. In fact, there was no evidence of crushing over the central support until the load was more than twice this value.

The measured span-moments were again in fair agreement with those calculated on the assumption of a constant support-moment after passing the elastic stage. Throughout the test the crack-widths were small (see Table I, p. 324) so that redistribution of

Fig. 8.



Calculated Stresses. (*Lb.-inch units; notation as given on p. 297*).

Stage.	Support.									Span.										Load W.	Dist. of point of inflexion from B.	Re-actions.	
	At B.									At D.						At A.							
	t	c	M	n	a	S	s	t_w	s_b	t	c	M	n	a	s_b	S	s	t_w	s_b			R_B	$R_A = R_C$
1	6,400	2,050	123,000	5·35	5·32	6,100	270	7,800	70	5,180	445	72,200	3·09	5·80	35	2,260	80	4,660	63	15,800	20·8	12,200	2,260
2			123,000			13,900	615	17,700	159	23,800	2,050	332,000	3·09	5·80	159	10,200	366	21,000	283	47,300	8·9	27,800	10,200

Dead-load effects.

M_B = 4,120 lbs.-inches. R_A = 200 lbs.
M_D = 910 lbs.-inches. R_B = 480 lbs.

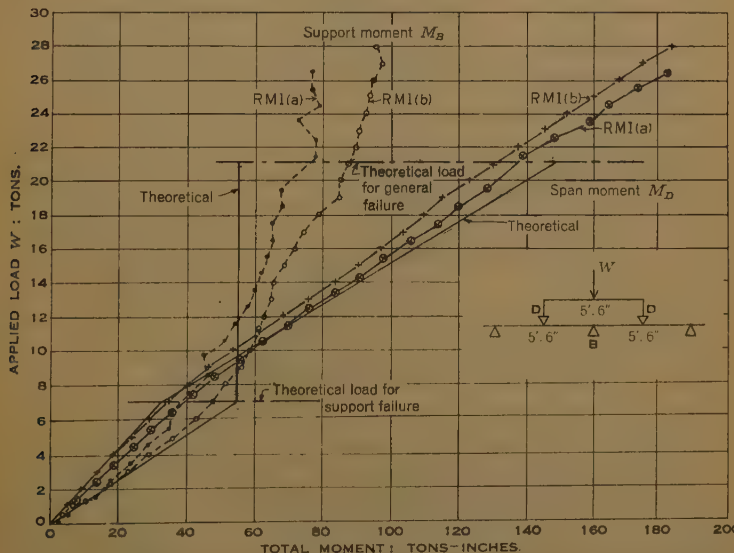
u = 2,050 lbs. per square inch. m = 9·5

CONTINUOUS BEAMS RM1 (a) AND RM1 (b): CONCRETE FAILURE.

moment in the case of concrete weakness may be considered without reference to cracking. The beam deflections were of the same order as those measured in the previous series.

(3) *Primary Concrete Failure: Compression Steel Provided over Central Support.*—In the tests designed to show weakness in compression in the presence of a limited amount of compression steel, the reinforcement was the same as in the previous beams, except that the lower bars were continuous throughout the beam, thus providing help in compression over the central support. Details of beams are given in *Fig. 10* (p. 304), the notation employed being as given on

Fig. 9.



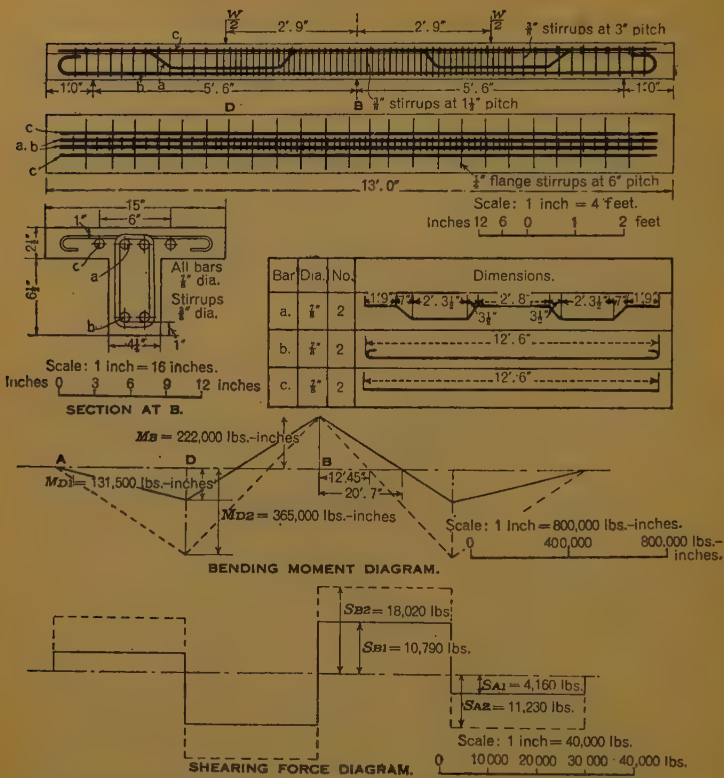
TESTS ON CONTINUOUS BEAMS. CONCRETE FAILURE (NO COMPRESSION STEEL).

p. 297, and t' denoting the stress in the longitudinal compression reinforcement. The concrete mix used was again $1 : 2\frac{1}{2} : 3\frac{1}{2}$ (by weight) using ordinary Portland cement, and the tests were made at an age of 7 days; the strength (Appendix I, p. 328) was a little higher than that obtained in the previous tests.

The moments measured during the test of one of the beams, RM3 (b), are given in *Fig. 11*, p. 305. Again there is a gradual change between the two stages of the test, and it is interesting that the final loads attained (Table II, p. 325) were almost exactly the same as for the beams in which no compression reinforcement was provided.

There was no evidence of compression failure over the central

Fig. 10.



Calculated Stresses (Lb.-inch units ; notation as on p. 297) RM3(a).

Stage.	Support.										Span.										Load W.	Dist. of point of inflexion from B.	Reactions.	
	At B.										At D.						At A.							
	t	t'	c	M	n	a	S	s	t_w	s_b	t	c	M	n	a	s_b	S	s	t_w	s_b			R_B	$R_A=R_C$
	15,100	14,400	2,250	222,000	4·43	6·09	10,790	416	12,000	161	9,470	812	131,500	3·09	5·80	169	4,160	149	8,600	115	29,000	20·7	21,580	4,160
				222,000			18,020	696	20,100	270	26,300	2,250	365,000	3·09	5·80	282	11,230	402	23,200	311	57,600	12·45	36,040	11,230

Dead-load effects.

M_B=4,120 lbs.-inches. R_A=200 lbs.
M_D= 910 lbs.-inches. R_B=480 lbs.

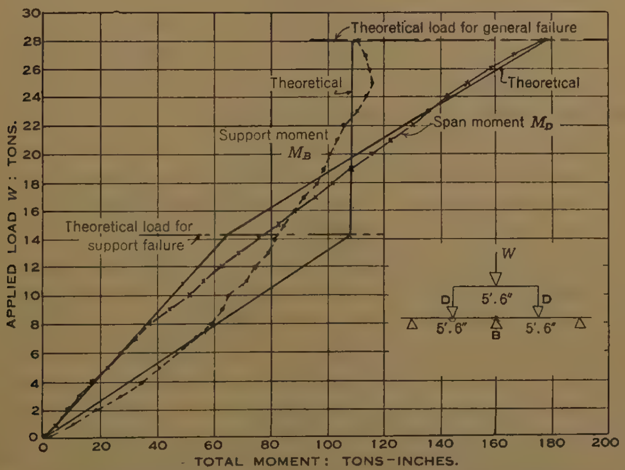
u=2,250 lbs. per square inch. m=9·5

CONTINUOUS BEAM RM3(a): CONCRETE FAILURE (WITH COMPRESSION STEEL).

support until just before final collapse of the system. The extent of the redistribution of stresses in the section over the central support can be judged from the fact that the main tensile crack at that section gradually closed towards the end of the test until it extended only about 2 inches from the top surface of the beam, indicating that the whole of the rib and even some of the flange were bearing compression forces. The drop in support moment near the ultimate load was probably due to bond failure of the compression bars. The maximum crack-widths are given in Table I, p. 324.

It is clear that, from the point of view of load-carrying capacity, the use of compression reinforcement in no way helped, although it

Fig. 11.



TEST ON CONTINUOUS BEAM RM3 (b); CONCRETE FAILURE, WITH COMPRESSION STEEL.

must be remembered that the tension reinforcement was decreased at the same time. It would appear, therefore, that in practice it would be wise to ignore the effect of the compression reinforcement in calculations of the strength of a continuous beam on the basis of redistribution of moments.

(4) *Primary Concrete Failure: Beams with Increased Span-Length.*—The beams in series (2) above were provided with closely-spaced stirrups over the central support in order to avoid shear failure with the weak concrete used. It was suggested that this reinforcement gave lateral support to the concrete, thus increasing its ability to carry longitudinal compression. In order to investigate whether this was the case, two further beams were prepared similar to those of series (2) except that the span length was increased to 12 feet, so

that the failing moments would be reached at a lower load, hence reducing the amount of shear reinforcement required.

The results showed conclusively that the central support section was not weakened by the wide spacing of the stirrups. The percentage increase in load due to redistribution was approximately the same as before (see Table II, p. 325), and the support moment carried at failure was actually greater than had been obtained in the previous tests of series (2).

(5) *Primary Concrete Failure at an Age of* $5\frac{1}{2}$ *Months.*—The tests previously carried out with weak concretes were made at an age of 7 days in all cases, and although it seemed probable that the amount of redistribution that occurred as a result of inelastic deformation of the concrete would depend on the strength of the concrete rather than its age, it was thought advisable to test two beams similar to those of series (2) (no compression reinforcement) at a greater age. In order to obtain a low strength at about 6 months an ordinary Portland cement was used in a mix of proportions 1 : 4 : 7 (by weight) for the first beam ; this was changed to 1 : 5 : 6 for the second to give a better mix with the same water—cement ratio of 1·05.

The failing loads, given in Table II, p. 325, were as great, and in one case greater, than those obtained previously. The concrete-strength was, however, not known very accurately, as the cubes cast with the beams could not be relied upon to give a fair estimate of the quality of the concrete in the beam itself for such a poor-quality concrete. Samples were cut from the ends of the beams and tested, and the results indicated that the concrete was, if anything, somewhat weaker than that used for the earlier tests. There is no doubt, therefore, that the redistribution obtained with the richer concrete was not attributable to the fact that it had hardened for only a comparatively short period.

Discussion of Results.

The tests at working-loads on built-in beams have shown that very little redistribution of moments can be expected as a result of creep of the concrete at working-stresses.

The actual failing-loads of the continuous beams, together with values calculated on various assumptions, are summarized in Table II, p. 325. It is apparent that with all the beams the ultimate load carried before failure of the system was greater than the theoretical load for support-failure calculated on the elastic theory. The increased load can be considered as due to two factors :—

(i) Redistribution of moments throughout the system, tending to give simultaneous failure both at the central support and in the span.

(ii) Redistribution of stress at the highly-stressed sections, increasing the moments these sections are capable of taking above the values as calculated by the ordinary theory.

In Table II the calculated loads are based on three sets of resistance moments calculated by different methods. The first is obtained by the use of the true or "instantaneous" modular ratio, that is, the ratio which neglects all creep of the concrete. The second is obtained by assuming that some stress-redistribution occurs at any section, corresponding to the use of a modular ratio $m = \frac{40{,}000}{\text{cube strength}}$, the value suggested for design-purposes in the "Recommendations for a Code of Practice for the Use of Reinforced Concrete in Buildings." The third set of resistance moments were calculated on the following assumptions for stress-redistribution :—

(*a*) In the case of primary tension-steel failure the steel will yield until the maximum concrete-stress reaches the cube strength of the concrete.

(*b*) In the case of primary concrete failure, the modular ratio will effectively increase to a value given by $m = \frac{80{,}000}{\text{cube strength}}$. If, however, tension-steel yield occurs when this higher value is used, the resistance moment is calculated as for (*a*). If the calculated stress in the compression steel exceeds its yield value when the higher modular ratio is used, the calculations are modified so that the compression bars do not exceed their yield value.

The use of this higher value for the modular ratio was suggested by strain measurements on simple and continuous beams and frames which indicated that failure of concrete in compression does not occur until a strain of from 25 to 35×10^{-4} is reached. On the assumption that failure occurs when the maximum stress is equal to the cube strength, the effective elastic modulus would be from $\frac{u}{2{,}500} \times 10^6$ to $\frac{u}{3{,}500} \times 10^6$, or the effective modular ratio from $\frac{75{,}000}{u}$ to $\frac{105{,}000}{u}$, where u denotes the cube strength of the concrete in lbs. per square inch. The value $\frac{80{,}000}{u}$ has been selected as representing sufficiently closely the lower limit, and because it is easily memorized as twice the usual value.

From Table II it will be seen that if the elastic theory is used for calculating the moments at failure, the theoretical failing loads are less than the actual ultimate loads even when allowance is made for

redistribution of stress. On the other hand, if redistribution of moments is allowed for, the theoretical loads for simultaneous failure at the central support and in the span, when no redistribution of stress is taken into account, are also less than the actual loads carried, though the margin of safety is not so great.

If allowance is made for redistribution of both moment and stress, the use of a modular ratio of $\frac{40,000}{u}$ leads to theoretical loads which are not greatly different from the actual ultimate loads, except in the case of the beams in which compression reinforcement was used over the central support with a weak concrete (series (3)). The use of the third method of full allowance for both moment and stress redistribution is clearly unsafe except in the case of primary steel-failure, for which it must be remembered that the redistribution of moments is accompanied by widening of the tension cracks (see Table I, p. 324).

The results of the tests on the beams in which compression reinforcement was provided are important. The use of a very high modular ratio for estimating the resistance moment of a section leads to increased computed stresses in the compression bars, and it does not appear advisable to rely upon this. In order to investigate this aspect more fully, some simple beam tests were carried out to measure the resistance moments of the sections similar to those used over the central support in the main tests; the results are given in Table III, p. 326. The resistance moments found in these subsidiary tests are compared with the values calculated on the same basis as before and with the values measured in the continuous-beam tests. It is apparent that the use of the highest modular ratio $\frac{80,000}{u}$ is reasonable in all cases of concrete failure, except those in which compression reinforcement was provided. In these cases the simple-beam tests indicated that redistribution of stress may occur to the extent indicated by the use of the lower modular ratio of $\frac{40,000}{u}$, whereas the support moments measured in the continuous-beam tests are not appreciably greater than those calculated on the basis of the "instantaneous" modular ratio. In the simple beams, serious shear cracking developed towards the end of the tests, and it is possible that the higher shear stresses in the continuous beams with compression reinforcement may have been the reason for the low moment carried over the central support. (It should be noted here that the support moments in the main beams were not measured directly but deduced from the end reactions calculated from the

strain in the supporting beam; any errors in the actual measurements are magnified in the calculation of the support moments, though not in the case of the span moments. The values given, therefore, for the main and the subsidiary beams may be considered in fair agreement, except in the case of the beams with compression reinforcement.) It appears, therefore, that when compression reinforcement is provided at the support its effect should be ignored in making calculations taking moment redistribution into account. If this is done for the present beams of series (3), the calculated loads are 28·9 and 31·6 tons, or 5 and 9 per cent. greater respectively than actually obtained. If the effect of the compression reinforcement in the span is also ignored, the calculated loads are 23·4 and 25·2 tons respectively, and these are on the safe side.

PART II.

Reinforced-Concrete Frames.

The investigation recorded in Part II of this Paper was made with the object of examining the necessity for designing columns to resist bending as well as direct stress. With this in view, tests were made to determine to what extent the load-bearing capacity of a simple reinforced-concrete portal frame may be increased as a result of redistribution of stress and moment when high stresses are reached at the column head.

The conditions tested were :—

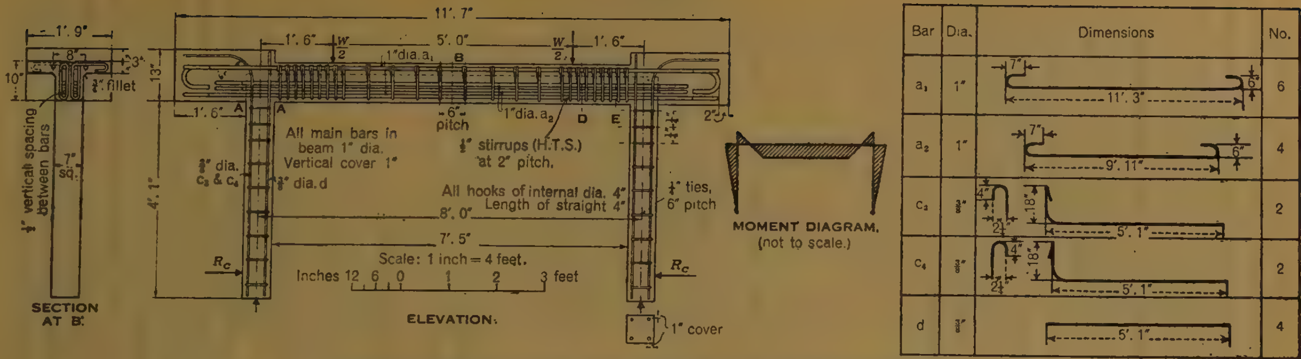
A. Primary failure of the tension steel in the column.
B. Primary failure of the concrete in compression in the column.

For each condition two frames were tested.

Primary Failure of Tension Steel in the Column.

Details of the frames and the positions of loading are given in *Fig. 12* (p. 310), the notation used for the stresses being as given on p. 297. The design of the reinforcement and the method of loading was such that the beam was considerably stronger than the column. The moments in the beam span for a particular load are, of course, increased as a result of increasing the stiffness, but this effect is very small compared with the actual increase in strength in the beam due to the added reinforcement. By applying the vertical load close to the columns, the beam moments have again been kept low, so that at incipient failure of the weak column there is a reserve of strength

Fig. 12.



Calculated Stresses. (*Lb.-inch units ; notation as given on p. 297*).

Stage.		Column.						Beam at B.			Beam at D.				Dist. of point of inflexion from column face.	Bond at E (lower bars).	Load W.	Reaction R_c.
		c	t	M_A	S	s	s_b	c	t	M_B	S	s	s_b	t_w				
1	$^*I_1$	4,200	47,300	102,000	2,380	65	190	920	7,000	206,000	17,300	415	155	7,600	2·6	190	34,500	2,380
	I_2	4,800	47,300	114,000	2,650	70	210	1,200	9,000	266,000	21,900	525	200	8,700	2·3	242	43,700	2,650
2		11,000	47,300	283,000	6,590	175	520	11,000	40,600	1,240,000	86,000	2,120	775	37,600	0·2	—	172,000	6,590

$u = 11{,}000$ lbs. per square inch. $m = 5$.

* I_1 Moments of inertia, for moment-distribution calculations, based on whole area of concrete ignoring steel.
I_2 Moments of inertia, for moment-distribution calculations, based on whole area of concrete including steel.

FRAMES RMF2 AND RMF3 : STEEL FAILURE.

in the beam, thus providing the best conditions for redistribution of moment.

In order to ensure that the frame should fail by bending, and not in shear or by slip of the bars, it was necessary to give special attention to the design of the shear reinforcement and the anchorage of the bars. It is clear that redistribution of moment can increase the ultimate load of a structure only when the conditions of bond and shear that result from such redistribution are amply provided for. The large blocks at the column—beam junctions were provided solely for the purpose of giving ample anchorage to the reinforcement of the beams and columns in order that the yield point of the steel could be reached.

A high-strength high-alumina cement concrete was used for these tests, details of which are given in Appendix II, p. 329.

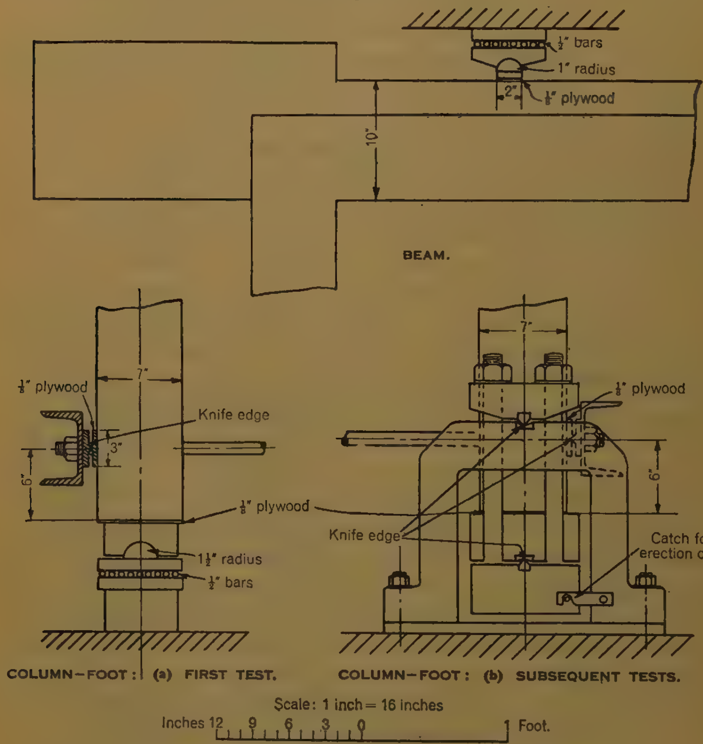
The horizontal load was applied by two helical tension springs stretched between the feet of the columns, the load being transmitted to the faces through knife-edges. The load on the beams was applied through cylindrical bearings and rollers to allow free rotation and translation of the beam. In the first test the feet of the columns were supported on similar bearings, but it was found that the frictional force due to the rollers was sufficient to affect appreciably the horizontal spring load required to prevent outward movement of the feet, and a special knife-edge link system was used for subsequent tests. Details of all loading arrangements are given in *Fig. 13* (p. 312).

During the test, gauges were set up at the feet of the columns to measure the movement outwards, and the horizontal load due to the springs was continually adjusted so that the feet were brought back to their original position. That is, the conditions of restraint were those of a portal, position-fixed and pin-jointed at the feet of the columns. A view of one of the frames whilst the test was in progress is given in *Fig. 14* (facing p. 312). A special framework was arranged to prevent any rotation or lateral movement of the supporting beam relative to the upper loading beam, so that no torsional stresses or lateral bending stresses should be set up in the columns.

As already stated, the horizontal forces at the feet of the columns were in the first test partly supplied by friction of the roller bearings used to support the frame, and the moments in the system were therefore not accurately known. The failing load was, however, over three times as great as the value which, according to the elastic theory, should produce column failure. It was clear that considerable redistribution had taken place, but for evidence as to how this occurred it is necessary to refer to the second test in which the special link bearings were used and the moments were accurately known throughout the test.

The main results for the second test are shown in *Figs. 15* and *16* (pp. 313–4). In *Fig. 15* the applied loads are plotted against the horizontal reactions which are proportional to the moments at the column head, and on the same Figure some theoretical curves are also given. One of these curves shows the load—reaction relationship expected for the frame from calculations based on the elastic theory;

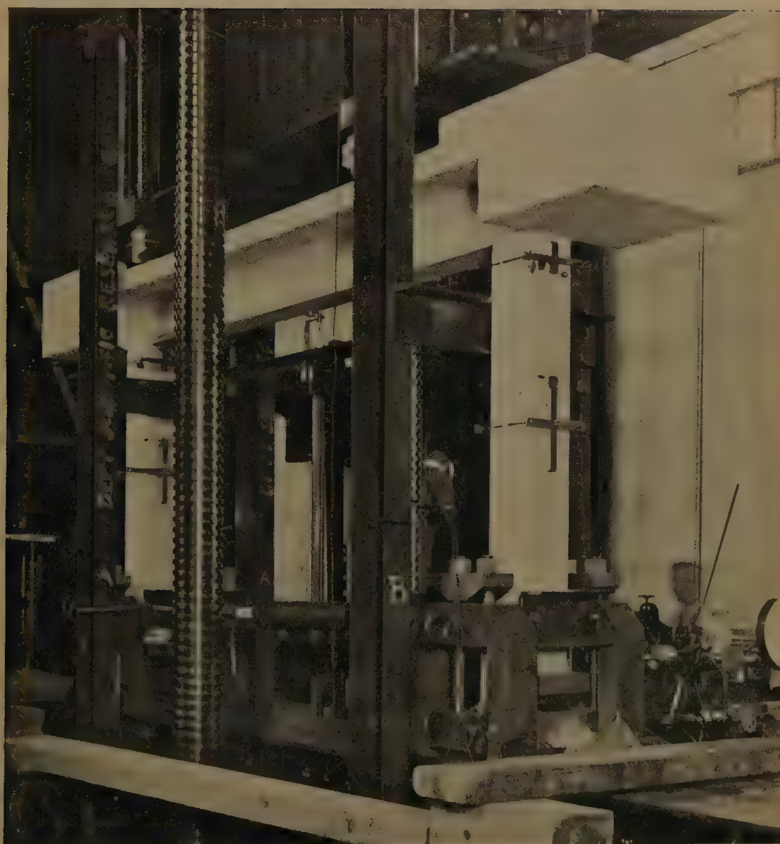
Fig. 13.



Loading Arrangements used in Frame Tests.

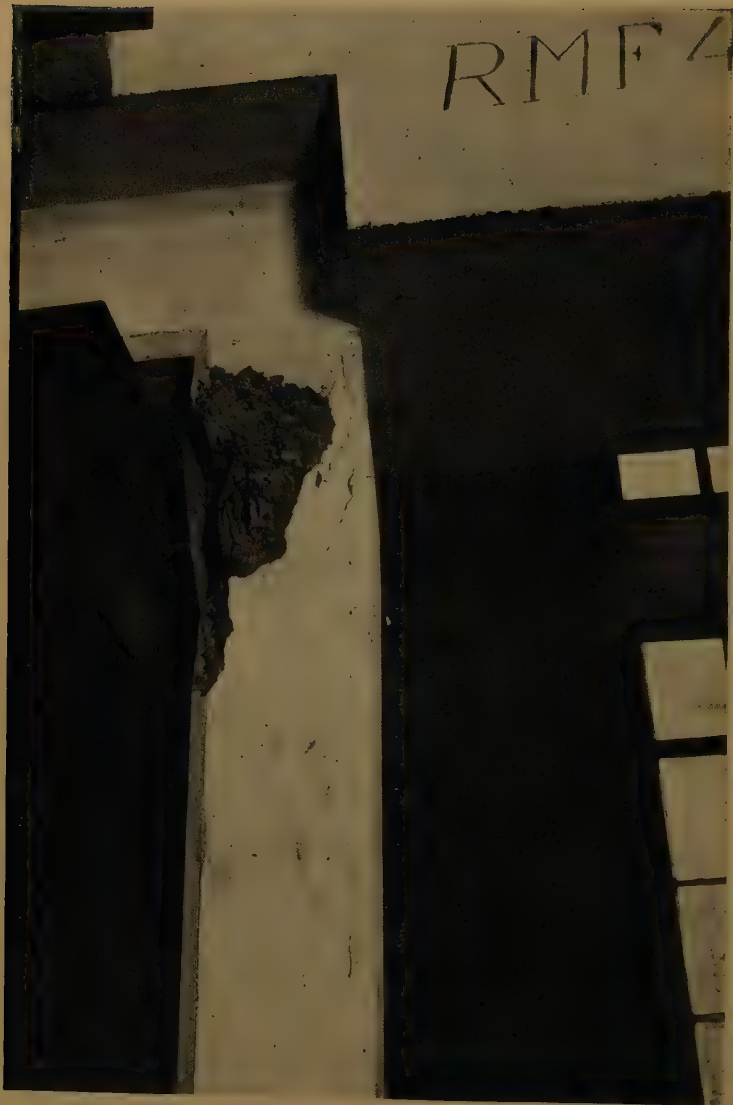
a series of curves are given for the relationship between the loads and reactions which produce steel-yield on the following assumptions:—(1) that the "instantaneous" modular ratio determines the stress distribution, (2) that the modular ratio is taken to be $m = \frac{40,000}{u}$, and (3) that the maximum concrete stress is assumed to reach the cube strength. The point where the first-mentioned curve intersects each of the steel-failure curves determines the load at which the frame should have failed according to the elastic theory, with or without

Fig. 14.



LOADING ARRANGEMENT FOR FRAME TESTS.

Fig. 20.

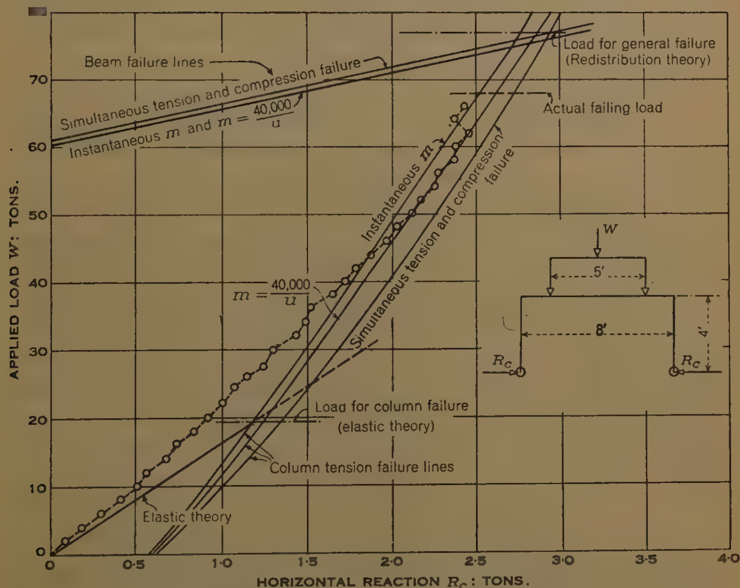


FRAME RMF4 (CONCRETE FAILURE) AFTER TEST.

allowance for stress redistribution according to which assumption the curve represents. These loads are given in Table IV, p. 327.

On the simplest theory of moment redistribution (that is, assuming that the column tension-steel remains continuously at its yield-point) the horizontal reaction and therefore the moments will, after yield of the column steel, conform to the relationship shown by one of the steel-failure lines in *Fig. 15*, according to the amount of stress-redistribution that occurs. The experimental results gave horizontal reactions which were initially somewhat lower than expected,

Fig. 15.



TEST OF FRAME RMF3: STEEL FAILURE.

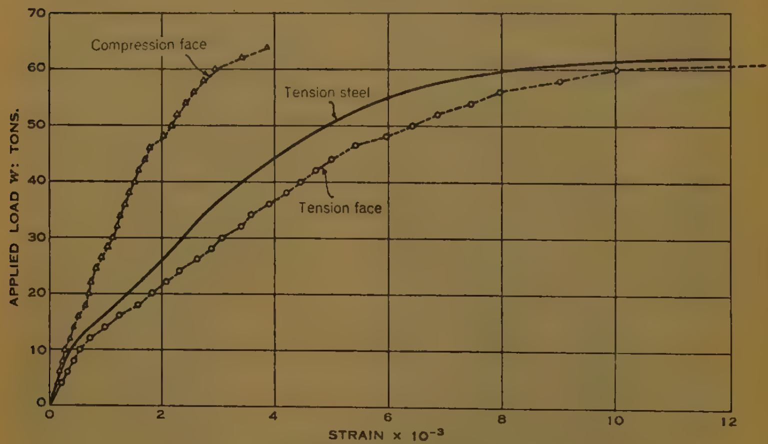
redistribution beginning at quite a low load soon after the appearance of cracks at the column head. The curve showing the experimental results gradually approaches the steel-failure lines as the load is increased and crosses the line based on $m = \frac{40,000}{u}$, showing that stress redistribution took place. Incipient concrete-failure caused a sudden drop in the rate of increase in moment, and finally failure was reached as a result of concrete crushing.

The beam-failure lines indicate the values for the applied load at which beam failure would occur for the degrees of fixity afforded by the various horizontal reactions, and it will be seen that if the

concrete in the column had not failed a slight increase in load could have been obtained before beam failure.

Throughout the test, measurements were made of the strains at the column heads; the results for one of the columns are given in *Fig. 16.* The strains were measured on the faces of the column; no direct readings were taken on the steel itself, the steel strain being deduced on the usual assumption that plane sections remain plane. Whilst this assumption cannot be expected to hold for the column head where cracks appear early in the test and shear deformations also occur, it is thought that the use of the assumption will not lead to very great error except in the final stages of the test. The strain for a steel stress of 47,300 lbs. per square inch (the yield stress, see

Fig. 16.



TEST OF FRAME RMF3: STEEL FAILURE.

Appendix II, p. 329) was reached at a load of just over 20 tons, and the strain increased to over four times this value before collapse became imminent. The concrete strain at first signs of crushing was about 32×10^{-4}, a value only a little higher than that obtained in a series of simple beam tests using a low-strength concrete.

The beam deflection was measured relative to the loading points by means of dial gauges. This deflection was only one-thousandth of the span at about three-quarters of the failing load. The overall longitudinal extension of the beam soffit was also measured; as failure of the frame was approached this movement was about $\frac{1}{12}$ inch at each column head. This movement is insufficient, as an added eccentricity, to have an appreciable effect on the stress at the column head.

Measurements of the widths of all cracks were made with a portable

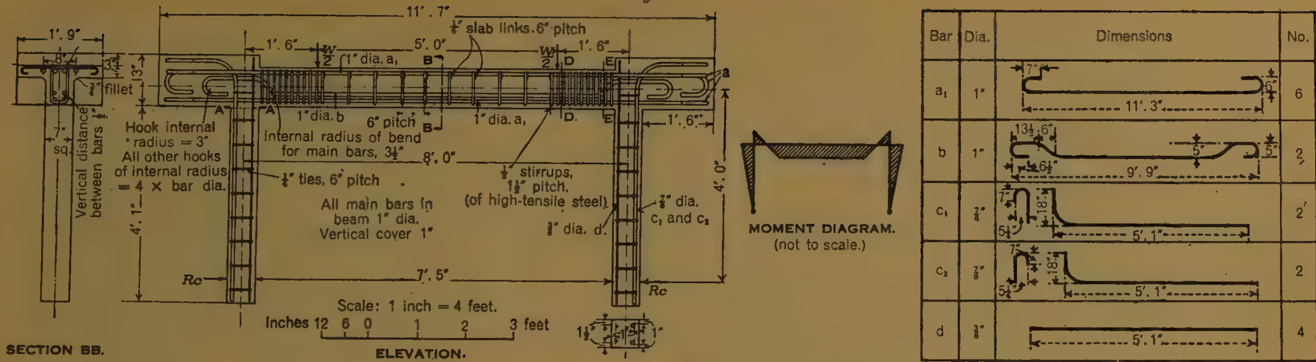
microscope. The cracks at the column head appeared at a load of about 5 tons, widened steadily throughout the test, and just before failure were about twice as wide as the cracks usually obtained when steel reinforcement reaches its yield point.

Primary Failure of Concrete in the Column.

Details of the reinforcement used for the second type of frame are given in *Fig. 17*, p. 316, the notation used for the stresses being as given on p. 297. Here again the design was arranged to give a reserve of strength in the beam. The tension steel in the column was increased to two $\frac{7}{8}$-inch diameter bars instead of $\frac{3}{8}$-inch bars, and the concrete used was an ordinary Portland cement of $1 : 2\frac{1}{2} : 3\frac{1}{2}$ mix to give a cube strength at the age of test (7 or 9 days) of between 2,000 and 3,000 lbs. per square inch. Details of the strengths of the steel and concrete are given in Appendix II, p. 329.

The method of test was identical with that used in the second frame of the previous series, and the results for the first frame are given in *Figs. 18* and *19* (pp. 317–8). Referring first to the moment diagram in *Fig. 18*, it will be seen that the initial relationship between vertical load and the horizontal reaction which is proportional to the moment at the column head is in good agreement with that expected from the elastic theory. According to this theory the concrete should crush at a load of about 21 tons, that is, at the load when the initial line in *Fig. 18* reaches the compression-failure line for a modular ratio of $m = 9$, which is the true value for the concrete used when inelastic deformations are disregarded. Curves representing compression failure are also given based on modular ratios of $\frac{40,000}{u}$ and $\frac{80,000}{u}$. It is evident from the Figure that the redistribution of stress in the column-head section is even more favourable than is assumed by this last line, probably due to the increased stresses taken by the concrete above those assumed by a linear distribution of stress from the neutral axis to the compressed face. Assuming, however, this last compression-failure line as a safe guide, it is seen that, unless moment redistribution occurs, there will be signs of distress in the concrete at a load of about 28 tons. If moment redistribution does take place, then the load will increase with a reduction of horizontal reaction until beam failure is reached at a load of about 48 tons. Actually, redistribution will start before signs of distress can be seen, and the approximate theoretical change in load and moment is indicated in *Fig. 18*. The actual curve shows that the theory is on the safe side, the moments increasing more than expected from the simple theory of redistribution, but with a sudden drop in moment after signs of crushing first appeared. The failing load, 47·1 tons, agrees well with the

Fig. 17.



Calculated Stresses. (*Lb.-inch units; notation as given on p. 297.*) RMF4.

Stage.		Column.						Beam at B.			Beam at D.				Dist. of point of inflexion from column face.	Bond at E (lower bars).	Load W.	Reaction R_c.
		c	t	M_A	S	s	s_b	c	t	M_B	S	s	s_b	t_w				
1	$*I_1$	2,850	15,000	127,000	2,960	90	115	860	12,300	245,000	21,500	465	260	12,400	3·1	460	43,000	2,960
	I_2	2,850	13,000	124,000	2,890	89	114	1,010	14,500	289,000	23,800	515	285	13,800	2·3	510	47,500	2,890
2		2,850	7,500	131,000	3,050	94	120	2,850	41,100	817,000	53,500	1,160	645	31,000	—	1,150	107,000	3,050

u = 2,850 lbs. per square inch. $m = 9$ (Stage 1). $m = 80{,}000/u$ (Stage 2).

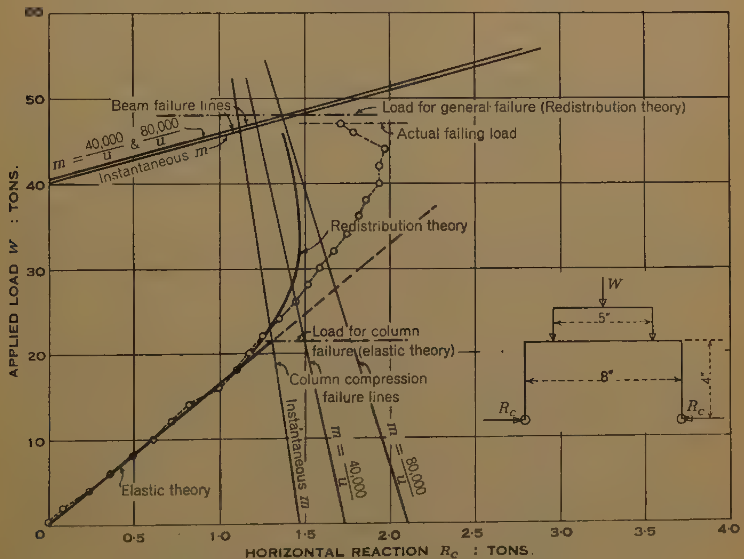
* I_1 Moments of inertia for moment-distribution calculations based on whole concrete area ignoring steel.
I_2 Moments of inertia for moment-distribution calculations based on whole concrete area including steel.

FRAMES RMF4 AND RMF5: CONCRETE FAILURE.

expected value (see Table IV, p. 327), and was the result of simultaneous crushing of the concrete in the column and yield of the steel in the beam.

The strains at the column head were measured as before, and are shown in *Fig. 19*. The steel strains have been interpolated, showing that the tension stresses were low throughout, but that the compression bars were working at their yield load towards the end of the test. These bars buckled at failure of the frame as shown in the photograph, *Fig. 20* (facing p. 313).

Fig. 18.



TEST ON FRAME RMF4 : CONCRETE FAILURE.

The deflection of the beam and the extension of the soffit were again small; the cracking of the column was also of little importance, whilst the beam cracks increased to a width of about 6 or 7 thousandths of an inch, a width usually associated with a steel-stress of about 40,000 lbs. per square inch.

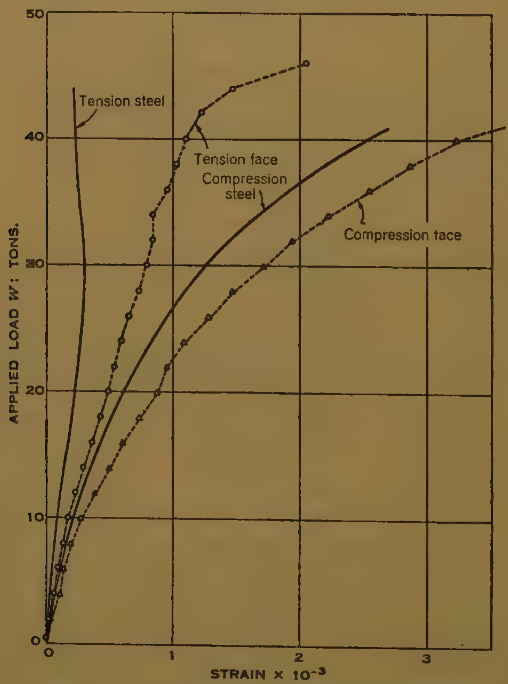
In the case of the second frame of this series the concrete-strength was somewhat less than that used for the first frame (see Appendix II, p. 329), but apart from the reduced values of load and moment due to this cause, the results were very similar to those already discussed. Again the use of a modular ratio of $\frac{80,000}{u}$, together with the assumption that the column will continue to deform so as to redistribute the

moments until beam failure occurs, leads to a close estimate of the conditions of failure (see Table IV, p. 327).

Discussion of Results.

It is clear from the tests that there may be considerable divergence between the actual ultimate load-carrying capacity of a frame and the load which, according to calculations based on the elastic theory, produces a stress in the concrete or steel, at the column head, equal to the ultimate strength of the concrete or the yield strength of the

Fig. 19.



TEST ON FRAME RMF4; CONCRETE FAILURE.

steel. It is important to note that in these tests special precautions were taken to prevent shear failure, closely-spaced high-tensile steel stirrups being provided in the beams, and special anchorage blocks at the beam—column junctions. Redistribution of moments cannot occur unless the secondary reinforcement and the anchorage of the steel are sufficient for the conditions resulting from the redistribution.

(1) *Steel Failure.*—In the case of primary steel-failure, the increase in load due to redistribution of moment and stress was over 200 per

cent. In this case, however, complete moment redistribution did not occur, and beam failure was not reached owing to the earlier crushing of the concrete in the column, even though the cube strength was 11,000 lbs. per square inch. In such cases it is not at present possible to calculate accurately the load at which the concrete will fail, since it depends on the deformation of the column after yield of the tension steel. The use of a reduced modular ratio may be permissible, but there is not yet sufficient evidence to show to what degree this reduction should be made. Since the extent to which redistribution can take place is, therefore, not clearly defined and redistribution leads to increased cracking, it would be wise to ignore it until further experimental evidence has been obtained.

The loads calculated for failure on various assumptions as to the extent of stress and moment redistribution are given in Table IV, p. 327. It will be seen that the effect of stress redistribution is to increase the load, when no moment redistribution occurs, by about 10 per cent. for $m = \frac{40,000}{u}$ and by about 30 per cent. when concrete-crushing is developed. The actual increase as shown by the position of the experimental points in *Fig. 15* corresponded approximately to an effective modular ratio of $m = \frac{40,000}{u}$.

It is interesting to compare the frames with the system of a beam resting freely on columns at its ends. The beam load at failure would then have been about 60 tons, so that the effect of the monolithic design of the portal is to increase the load-carrying capacity of the beam by about 10 per cent. The strength of one of the columns in axial compression was over 150 tons, whereas the actual load carried in the frame tests was only just over 30 tons for column failure. That is, the load-carrying capacity of the columns was reduced to one-fifth of their full strength as a result of the bending moments induced by frame action. The frames tested were designed for extreme conditions of bending in the columns, and, although the results prove that such bending can seriously reduce their strength, in most practical cases the effect would be very much less.

(2) *Concrete Failure.*—As in the case of primary steel-failure, there are considerable increases in the ultimate loads carried by the frames as a result of redistribution of stress and moment. If it is considered that the useful limit of increase of load is when signs of crushing first appear on the column faces, it will be seen from Table IV that the load increase above the value calculated on the elastic theory was 90 per cent. for the first frame and 150 per cent. for the second frame. In both cases the increase in beam load-carrying capacity as a result of the column moment was less than 20 per cent., whereas the

columns would, if loaded axially, have been able to withstand about twice the load that they took in the frame test. Again the need for taking bending in columns into account is evident.

An estimate of the effects of redistribution can be made in simple cases where concrete-failure is the deciding factor on the following assumptions :—

(i) that the modular ratio can be taken as $\frac{80,000}{u}$,

(ii) that both column head and span develop their full strengths before failure of the system occurs.

In any cases where the use of the higher modular ratio leads to calculated stresses in the tension steel greater than the yield point of the steel, the particular section should be calculated on the assumption that both steel-yield and the full concrete-strength are developed.

It is clear from *Fig. 18* that stress redistribution occurred in the column-head section to a greater extent than that indicated by the use of a modular ratio of $\frac{80,000}{u}$, and from this figure and Table IV (p. 327), it is seen that the effect of stress redistribution, if moment redistribution is ignored, is to increase the failing load by at least 30 per cent. for the particular section used. The increase may not be so great in other cases. For example, in the continuous-beam tests described in Part I of this Paper the increase in resistance moment due to stress redistribution was only about 13 per cent. for the central support section of the beams of series (2) and (4) (see Table III, p. 326). The smaller amount of stress redistribution that occurred in beam sections reinforced in compression must also be borne in mind, and it would therefore be unwise to calculate the compression steel for stresses up to the values given by $\frac{80,000}{u}$.

CONCLUSIONS.

Part I.

It has been shown that considerable moment redistribution can occur in continuous beams, failure of the system resulting from simultaneous failure of both support and span sections. In cases where the tension steel is the deciding factor, the effect of stress redistribution is to develop the cube strength of the concrete at the compression face before failure. This may be taken into account together with full allowance for moment redistribution when calculating the load-carrying capacity of a continuous beam system.

Since moment redistribution in cases of steel weakness leads, however, to increased cracking, the presence of these cracks and the possible risk of corrosion must be seriously considered before adopting this method in design. In the case of beams of high span—depth ratios in which the change in end slopes between two adjacent spans is necessarily considerable in order to enable full moment-transfer, it is possible that the full redistribution may not be developed.

Where no compression reinforcement is provided at the support, a fair estimate of the failing load can be obtained by considering that full moment redistribution occurs and that the resistance moments of the principal sections are those calculated on the basis of a modular ratio of $m = \frac{40,000}{u}$ as recommended in the Code of Practice. Where compression reinforcement is used at the support, the load may be similarly calculated but the effect of the compression reinforcement should be ignored. It would be wise to ignore the effect of compression steel both at the support and in the span, and to adopt a somewhat higher factor of safety against failure than when the ordinary elastic theory is employed.

Moment redistribution should not be allowed for by assuming higher stresses in design. In a beam designed to conform at all points with the moments calculated on the elastic theory, it is unlikely that there will be any important redistribution of moment, since failure will occur simultaneously at all points of the system.

The tests carried out indicate the safety of the recommendation in the Code of Practice that " provided the maximum positive moments in any two adjacent spans are increased by an amount not exceeding 15 per cent. of the maximum intermediate support moment, this latter may be reduced by the same amount and the bending moment curves adjusted accordingly."

It is important that both shear and bond stresses should be carefully considered in designing continuous beams for redistribution of moment. For example, where steel is required to work at its yield point, lower bond stresses than normally used should be allowed and anchorage by hooks or other mechanical means is desirable.

Part II.

The tests carried out on portal frames indicate that redistribution of moment and stress will occur in cases where the column is weaker than the beam. The ultimate load-carrying capacity of the frame will be increased beyond the value at which, according to the elastic theory, the stress in the concrete or steel at the column head reaches the ultimate strength of the concrete or the yield-strength of the

steel, by an amount depending on the relative stiffness of the beam and column, and on the amount of deformation possible in the weaker section.

Where the weaker section is capable of sufficient deformation, redistribution tends to develop the full strengths of both the beam span and the column-head sections. In the case where steel-yield occurred, the weaker section was incapable of sufficient deformation so that the ultimate beam-strength was not reached and the principle of simultaneous failure at both parts of the frame did not hold.

It is adequately demonstrated that redistribution of moments can easily have assisted materially in rendering safe many buildings designed without consideration of the bending moments in the columns. Although it is hoped as the result of further investigation to fix safe limits of deformation, in view of the lack of evidence at the moment it would appear wise to continue to make allowance for bending moments in columns, as required by the Code of Practice.

General.

There are three methods of design of reinforced-concrete framed structures :—

(*a*) by assuming continuity between certain members and discontinuity between others (for example, considering beams and slabs as continuous over supporting columns about which they can freely rotate) ;

(*b*) by assuming continuity throughout and designing all sections for the loads and moments calculated according to the elastic theory ;

(*c*) by assuming continuity throughout but allowing for the redistribution of moments : that is, the moments used in design for the various sections differ somewhat from those calculated by the elastic theory, although the sum of the moments adopted is roughly the same.

The tests have shown that in cases where method (*a*) is used in design, the disregard of continuity in certain parts of the construction may be compensated for by the redistribution of moments, which has the effect of increasing the ultimate loads of the structure beyond the load at which the weakest section would theoretically fail according to the elastic theory.

In certain cases the design of frameworks on the basis of redistribution of moments (method (c) above) would undoubtedly prove satisfactory. The work has indicated, however, that design on this basis must take into account the higher bond and shear stresses that accompany redistribution. At the same time the cracking will

usually be greater, particularly where the amount of tensile steel is reduced below that required by the elastic theory, and no great allowance for redistribution of moments should therefore be made where such cracking is likely to be harmful.

The limits of deformation that can occur in a section after steel yield without collapse of the section have yet to be determined, and, until this has been done, it would appear wise that the design of reinforced concrete frameworks should not deviate greatly from the requirements of the elastic theory. In any case, it appears that design using the principles of redistribution of moments is only likely to be justified in practice where special considerations involving difficulties in design or construction make it unprofitable to use the elastic theory.

The work described in this Paper forms part of the programme of the Building Research Board carried out in co-operation with the Reinforced Concrete Association. It is published with the permission of the Department of Scientific and Industrial Research.

The Paper is accompanied by eighteen sheets of blue-prints and two photographs, from which the Figures in the text and the half-tone page plates have been prepared, and by the following four Tables and two Appendixes.

The Council invite written communications on the foregoing Paper, which should be submitted not later than 2 months after the date of publication. Provided that there is a satisfactory response to this invitation it is proposed, in due course, to consider the question of publishing such communications, together with the Author's reply.

TABLE I.—MAXIMUM CRACK-WIDTHS IN CONTINUOUS BEAMS.

Series.	Load: tons.	Maximum crack-width: inch × 10^{-3}. Over central support. 5	10	15	20	25	Yield.*	In span. 5	10	15	20	25	Yield.*
1. Steel failure	(a)	0	15	30	42	60	0	0	1·3	2·3	2·6	3·8	0
	(b)	6	15	34	55	79	5	0	1·5	2·6	4·6	6·6	0
2. Concrete failure (no compression steel)	(a)†	0	1·5	2·4	3·1	3·3	0·5	0	1·9	3·5	6·0	10·0	0·6
	(b)	0	1·3	2·2	2·6	2·6	1·2	0	1·3	2·2	3·3	3·9	0·7
3. Concrete failure (with compression steel)	(a)	1·0	3·1	3·7	4·6	5·5	3·4	0	0·9	1·6	2·4	3·5	1·3
	(b)	1·6	4·0	5·2	9·2	10·5	4·8	0	1·3	1·7	2·6	5·2	1·5
4. Concrete failure (increased span-length)	(a)	3·3	3·7	—	—	—	1·6	1·5	4·0	—	—	—	0·8
	(b)	0·1	1·0	—	—	—	0	1·3	4·2	—	—	—	0
5. Concrete failure (weak concrete at about 6 months)	(a)	0	1·6	2·7	2·6	1·5	—	0	1·3	2·5	3·6	5·0	—
	(b)	0	0·7	1·0	1·1	1·2	—	0	1·4	2·4	3·5	7·2	—

* The yield load is the theoretical load for support failure according to the elastic theory (see Table II).

† The maximum crack-width in beam (a) of series (2) was measured at the depth of the most highly-stressed edge of the tension steel; in all other beams the measurements were made at the depth of the centre of the most highly-stressed bar.

TABLE II.—FAILING-LOADS OF CONTINUOUS BEAMS.

Basis of bending-moment calculations.	Basis of resistance-moment calculations.		Failing-loads: tons. 1. Steel failure.		2. Concrete failure (no compression steel).		3. Concrete failure (compression steel).		4. Concrete failure (increased span length).		5. Concrete failure (age 5½ months).	
		Test No.:—	RM2 (*a*).	RM2 (*b*).	RM1 (*a*).	RM1 (*b*).	RM3 (*a*).	RM3 (*b*).	RM4 (*a*).	RM4 (*b*).	RM5 (*a*).	RM5 (*b*).
Elastic Theory. (No redistribution of moments. Loads are for support failure.)	No stress redistribution.	True "instantaneous" modular ratio used . .	4·9	4·9	7·0	7·2	13·0	14·2	2·7	2·3		
	Stress redistribution.	$m = \dfrac{40{,}000}{\text{cube strength}} = 40{,}000/u$	5·0	4·9	7·6	7·8	19·5	19·8	3·0	2·5		
		Steel failure: maximum concrete stress reaches cube strength. Concrete failure: $m = 80{,}000/u$	7·8	6·5	8·0	8·2	25·4	26·2	3·2	2·7		
Theory of Redistribution of Moments. (Simultaneous failure at central support and in span.)	No stress redistribution.	True "instantaneous modular ratio used.	22·7	22·6	20·8	21·4	25·7	28·1	9·8	8·6		
	Stress redistribution.	$m = 40{,}000/u$ . .	23·0	22·6	27·8	28·5	35·0	36·3	13·0	11·8		
		Steel failure: maximum concrete stress reaches cube strength. Concrete failure: $m = 80{,}000/u$	26·1	24·0	32·6	32·8	40·1	40·5	14·2	13·9		
Actual load at which signs of distress were first noticed in the concrete			—	—	20·8	24·0	23·0	24·0	9·0	9·5	18·8	16·5
Actual ultimate load carried by beam			29·1	28·7	27·5	28·6	27·6	28·9	13·4	13·0	33·0	27·5

TABLE III.—RESISTANCE MOMENTS OF CENTRAL SUPPORT SECTIONS.

Method of obtaining resistance moment.	Resistance moments of support sections: * tons-inches.							
	1. Steel failure.		2. Concrete failure (no compression steel).		3. Concrete failure (compression steel).		4. Concrete failure (increased span length).	
	RM2 (*a*).	RM2 (*b*).	RM1 (*a*).	RM1 (*b*).	RM3 (*a*).	RM3 (*b*).	RM4 (*a*).	RM4 (*b*).
Theoretical. $m =$ " Instantaneous " Value	33	31	54	56	99	109	57	49
Theoretical. $m = \frac{40,000}{u}$	34	31	59	60	148	150	62	54
Theoretical. Steel failure: maximum concrete stress reaches cube strength. Concrete failure: $m = \frac{80,000}{u}$	52	43	61	63	192	198	65	56
Measured. Simple-beam test	58	56	103	104	145	159	108	93
Measured. Continuous-beam test	54	37	79	97	101	115	125	131

* By " resistance moment " in these Tables is meant the ultimate moment that the section can carry.

TABLE IV.—FAILING-LOADS OF PORTAL FRAMES.

Basis of bending-moment calculations.	Basis of resistance-moment * calculations.		Failing-loads: tons. Steel failure.		Concrete failure.	
		Test No.:—	RMF2.	RMF3.	RMF4.	RMF5.
Elastic Theory. (No redistribution of moments. Loads are for column head failure.)	No stress redistribution.	True "instantaneous" modular ratio used	19·5	19·5	21·2	15·0
	Stress redistribution.	$m = \frac{40,000}{\text{cube strength}} = \frac{40,000}{u}$ . .	21·3	21·3	24·0	18·3
		Steel failure: maximum concrete stress reaches cube strength. Concrete failure. $m = \frac{80,000}{u}$	25·0	25·0	27·5	21·4
Theory of Redistribution of Moments. (Simultaneous failure at column head and in beam span.)	No stress redistribution.	True "instantaneous" modular ratio used	75·0	75·0	46·0	41·7
	Stress redistribution.	$m = \frac{40,000}{u}$	75·5	75·5	46·8	42·6
		Steel failure: maximum concrete stress reaches cube strength. Concrete failure: $m = \frac{80,000}{u}$	77·0	77·0	47·8	43·6
Actual load at which signs of distress were first noticed in the concrete			65·0	64·0	40·0	38·0
Actual ultimate load carried by frame			65·0	67·8	47·1	43·2

* By "resistance moment" in these Tables is meant the ultimate moment that the section can carry.

APPENDIX I.

Quality of Concrete and Steel used in Connection with Built-in and Continuous Beam Tests.

(a) Concrete.

Series.	Beam.	Concrete mix (by weight).	Water/cement ratio (by weight).	Age at test.	Cube strength: lbs. per square inch.	True "instantaneous" modular ratio.
Built-in beams	1	P. 1 : 2 : 4	0·60	12 days	2,710	8·6
	2	P. 1 : 2 : 4	0·60	13 days	2,910	8·3
1. Steel failure	RM2 (*a*)	H.A. 1 : 2 : 4	0·60	6 days	10,140	5·0
	RM2 (*b*)	R.H.P. 1 : 1 : 2	0·44	44 days	6,660	6·0
2. Concrete failure (no compression steel)	RM1 (*a*)	P. 1 : 2½ : 3½	0·66	7 days	2,020	10·0
	RM1 (*b*)	P. 1 : 2½ : 3½	0·66	7 days	2,070	10·0
3. Concrete failure (with compression steel)	RM3 (*a*)	P. 1 : 2½ : 3½	0·66	7 days	2,250	9·5
	RM3 (*b*)	P. 1 : 2½ : 3½	0·66	7 days	2,470	9·1
4. Concrete failure (increased span length)	RM4 (*a*)	P. 1 : 2½ : 3½	0·66	7 days	2,130	9·7
	RM4 (*b*)	P. 1 : 2½ : 3½	0·66	7 days	1,830	10·4

P. = ordinary Portland cement. H.A. = high-alumina cement. R H.P. = rapid-hardening Portland cement.

(*b*) *Steel.*

Series.	Bar diameter: inch.	Yield stress: lbs. per square inch.*	Failing stress: lbs. per square inch.*
1. Steel failure	$\frac{5}{8}$ $\frac{3}{8}$	39,400 44,700	— 62,200
2. Concrete failure (no compression steel)	$\frac{7}{8}$ $\frac{3}{8}$	40,200 46,100	56,500 61,500
3. Concrete failure (with compression steel)	$\frac{7}{8}$ $\frac{3}{8}$	39,800 46,700	53,800 62,700
4. Concrete failure (increased span length)	$\frac{7}{8}$ $\frac{3}{8}$	37,900 46,700	53,300 61,800
5. Concrete failure (weak concrete at about 6 months)	$\frac{7}{8}$ $\frac{3}{8}$	36,600 45,800	51,500 61,400

* The stresses are in all cases based on the nominal original area of the bar.

APPENDIX II.

Quality of Concrete and Steel Used in Connection with Portal Frame Tests.

(*a*) *Concrete.*

Series.	Beam.	Concrete mix (by weight).	Water/cement ratio (by weight).	Age at test.	Cube strength: lbs. per square inch.
Steel failure . .	RMF2 RMF3	H.A. 1 : 2 : 4 H.A. 1 : 2 : 4	0·60 0·60	48 days 4 months	10,500 11,000
Concrete failure .	RMF4 RMF5	P. 1 : $2\frac{1}{2}$: $3\frac{1}{2}$ P. 1 : $2\frac{1}{2}$: $3\frac{1}{2}$	0·66 0·66	9 days 7 days	2,850 1,850

P. = ordinary Portland cement. H.A. = high-alumina cement.

(*b*) *Steel.*

Series.	Beam.	Bar diameter: inch.	Yield stress: lbs. per square inch.*	Failing stress: lbs. per square inch.*
Steel failure . . .	RMF2	$\frac{3}{8}$ 1 $\frac{1}{2}$†	49,200 41,500 66,900	60,800 63,700 106,000
	RMF3	$\frac{3}{8}$ 1 $\frac{1}{2}$†	47,300 40,600 63,800	59,700 65,700 107,000
Concrete failure . .	RMF4 & RMF5	$\frac{7}{8}$ 1 $\frac{1}{2}$† $\frac{3}{8}$	38,600 41,100 64,700 48,300	53,800 63,000 107,000 60,300

* The stresses are in all cases based on the nominal original area of the bar.

† High-tensile steel used for web reinforcement of beam.

Paper No. 5032.

"The Construction of Infiltration Galleries for the Water-Supply of Ndola, Northern Rhodesia."

By Robert Cunningham McIlleron, B.Sc., Assoc. M. Inst. C.E.

(*Ordered by the Council to be published in abstract form only.*)

It had been decided, in view of the uncertain demand, to obtain the water-supply for the town of Ndola from infiltration galleries. The shaft would give access to a series of galleries, which could be extended as necessary. It was decided to sink the shaft inside a reinforced-concrete cylinder with a cast-iron cutting-edge in order to remove the possibility of the walls collapsing, and also to prevent surface water from flooding the shaft.

The cylinder, which had an internal diameter of 8 feet, was sunk to a depth of 70 feet by adding concrete at the surface as the sinking proceeded, and the shaft was further extended in solid rock. The whole of the material penetrated had to be picked away as it was not possible to use grabs; as a result, the cylinder had to be kept dry by pumping and, as the depth increased, mud-rushing became more and more serious. The final level of the water entering the cylinder varied between 4 feet and 10 feet below ground-level, as the site of the cylinder was adjacent to a swamp; any failure in the pumping plant resulted in the pumps being submerged, and all operations had then to cease until the pumps had been hauled to the surface, restarted and again lowered as the water-level dropped.

The cause of the serious mud-rushing was attributed to the progressive collapse of a saturated stratum which was reached at 25 feet below the surface, while the difficulty of dealing adequately with the situation was attributed to the type of steam sinking-pump which was used. Ndola is 1500 miles north of Johannesburg, the nearest centre where adequate plant could readily be obtained, and, on account of the delay and expense involved in altering the existing equipment or in obtaining new plant, the work proceeded with the machinery available.

Mud-rushing was not anticipated, as a pilot borehole which was taken to a depth of 200 feet did not require a lining, even when it was pumped out. Apparently the strata for 50 feet below the surface consisted of well-compacted water-laden layers; these layers,

however, were liable to disintegrate after a time when water was induced to flow through them to an excavated face, and it was this continuous disintegration in the stratum at a depth of 25 feet which eventually caused the materials above to subside into the cavity so formed. This caused a large mass of loose, saturated materials to rush, without warning, into the pumped-out cylinder.

The Paper describes how a mud-rush occurred when the cutting-edge had been carried down into firm rock, and after the pump-house had been built. This mud-rush caused a subsidence below the foundations of the pump-house and created a dangerous situation, which was overcome by pumping cement-grout first into cavities below the floor and foundations, and then into the space between the cylinder-wall and the excavated rock near the cutting-edge, thus replacing the foundations of the building and filling all cavities in which mud could flow.

Considerable difficulty was experienced in getting the cylinder to sink, both weighting and using jets of water proving ineffective. The method finally adopted was to allow the cylinder to fill with water to its normal level and then to explode one or two sticks of gelignite under water. The water-hammer so induced apparently had the effect of expanding the cylinder, and side-friction became reduced. It was found that this procedure invariably induced sinking to take place.

In conclusion, it is pointed out in the Paper that mud-rushing might have been avoided by obtaining conditions where the cylinder sank rapidly and continuously, as there would not have been time for any extensive disintegration of the strata to have taken place. The steam sinking-pumps used, although of the plunger type designed to deal with muddy water, could not be worked efficiently owing to the rapidity with which their packing wore out, and it was realized that the pumps which should have been used were of the "pulsometer" or pistonless type, and that the aim should have been to sluice out the excavated materials by means of such pumps. Had this been the aim in the first instance it might have been possible to sink the cylinder more rapidly, and much of the difficulty encountered might have been avoided.

The shaft was eventually carried down to 80 feet below the surface of the ground, and a horizontal gallery was then driven for a distance of 28 feet. At this point a fissure was encountered from which the flow was 15,000 gallons per hour, and as this was sufficient for the immediate needs of the town, further tunnelling was unnecessary.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Sub-Committee on Velocity Formulas for Open Channels and Pipes.

Many formulas have from time to time been proposed to express the relation between hydraulic gradient and velocity in open channels and pipes. They are largely empirical in character, and of limited range, each being based upon particular series of experimental results. Recent researches abroad have indicated a new line of theoretical approach by which the flow of all fluids in channels or pipes of any size can be embraced in a single formula containing a roughness constant dependent only upon the nature of the bounding surface. This constant is a function of the size of the surface protuberances, but further research is necessary before such roughness sizes can be stated with certainty for all of the wide variety of surfaces found in practice. The problem is complicated by the fact that at sufficiently low velocities all pipes behave as if they were perfectly smooth, and so records from experiments under such conditions can yield no information concerning the nature of the pipe surface. A study is required of the resistance to flow in the range intermediate between completely rough and completely smooth. A closely allied question is the effect of age on loss in carrying capacity of pipes. With open channels the question of bed stability arises, particularly as contradictory data are on record as to the velocity necessary to avoid silting and scouring.

It has been decided to appoint a Sub-Committee of the Research Committee to investigate this problem. The constitution is as follows:—

Mr. W. J. E. Binnie (Chairman),
Mr. William Allard,
Mr. A. A. Barnes,
Mr. B. W. Bryan,
Mr. J. R. Davidson,
Prof. A. H. Jameson,
Mr. J. M. Lacey,
Mr. W. N. McClean,
Mr. R. W. S. Thompson,
Dr. C. M. White.

Two panels have been formed: the first consisting of

Prof. A. H. Jameson (Chairman),
Mr. A. A. Barnes,
Dr. C. M. White,

to evolve a basic formula and suggest research if necessary, and the second consisting of

Mr. J. M. Lacey (Chairman),
Mr. William Allard,
Dr. C. M. White,

to collect information relating to channels, including the effect of silt in suspension.

A questionnaire is being circulated to various water authorities seeking information relating to the results of tests on flow in pipes and pipe-lines, and it is hoped that members who have such information will place it at the disposal of the Sub-Committee.

Sub-Committee on Earth-Pressures.

The research work on earth-pressures to which The Institution is contributing, and which is in hand at the Building Research Station, includes a study of the shear strength of clay soils. The following note by Messrs. L. F. Cooling, M.Sc., and D. B. Smith, B.A., summarizes the more recent work on this aspect.

THE SHEARING RESISTANCE OF SOILS.

Introduction.

In the majority of foundation problems it is important that the differential settlements of the various parts of the structure should be quite small, and, since soil-strata in general are not completely uniform over the whole site of a building, foundation loads are kept well below the ultimate supporting power of the soil. There are cases, however, where there is danger that actual rupture of the soil may occur, as for example in engineering structures such as dams, retaining walls, and cuts and fills where the load distribution is unsymmetrical and there is a lack of lateral support. For the analysis of such problems a knowledge of the shear strength of the soil is of prime importance.

In common with other soil-properties the shear strength is mainly influenced by the water-content but it is also affected by the "structure" or the way in which the soil-particles are arranged. Its determination is further complicated by the fact that resistance to shear is itself a complex property. It is usually considered that

resistance to shear depends on the combined effect of two physical properties—cohesion and internal friction, the contribution due to cohesion being independent of any externally applied pressure. This hypothesis may be expressed in symbols by the modified Coulomb relation

$$t = c + n \tan \phi \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which t represents the resistance to lateral displacement along an interface, c the resistance due to cohesion, n the normal component of the pressure on the interface, and ϕ the angle of internal friction.

To analyse practical problems it is essential to be able to separate these physical properties and assess the contributions made by each factor in providing the total resistance. There are two quite different methods for calculating c and ϕ; one by deduction from measurement of actual earth-slide failures and the other by certain laboratory tests. A summary of values of c and ϕ for different kinds of soils collected from the work of various experimenters has been given by K. Terzaghi.[1]

In the laboratory shearing resistance is usually determined by tests in a shearing box which holds the soil in a completely-restrained state and measurements are made of the force required to slide a section of the soil-sample in a direction at right angles to an applied pressure. From a series of such tests performed under different normal pressures, values of c and ϕ have been deduced. This method has been used by Jacquinot and Frontard,[2] A. L. Bell,[3], H. Krey [4] and others. As Terzaghi has pointed out, the application of a normal pressure to a completely-restrained soil-sample tends to force water out of it, and if water is escaping during the test it influences the results considerably. Terzaghi,[1] Casagrande, and Gilboy [5] performed shear tests in an apparatus with porous ends, allowing sufficient time for the water to be extruded and hydrostatic equilibrium to be attained. Apart from consideration of movement of water, however, this type of test has the further disadvantages that the stress distribution is of a complex nature and that the shear stress probably varies along the dividing plane.

Other types of test have been tried, and among them is an

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[1] K. Terzaghi, "The Mechanics of Shear Failures on Clay Slopes and the Creep of Retaining Walls," Public Roads, vol. 10 (1929), p. 177.

[2] J. Résal, "Poussée des Terres," deuxième partie, p. viii, Paris, 1910.

[3] A. L. Bell, "The Lateral Pressure and Resistance of Clay and the Supporting Power of Clay Foundations," Minutes of Proceedings Inst. C.E., vol. 199 (1915), p. 233.

[4] H. Krey, "Erddruck, Erdwiderstand und Tragfähigkeit des Baugrundes," Berlin (1926).

[5] G. Gilboy, "Soil Mechanics Research," Trans. Am. Soc. C.E., vol. 98 (1933), p. 218.

interesting method described in an abstract of a Paper by A. L. Higgins [1] in which he measured shearing resistance by twisting an annulus of clay under different normal pressures in an almost completely-restrained state.

The present note describes an attempt to measure the shearing resistance of clay soils under simple conditions of stress and with minimum restraint. This has been done by a torsional shear apparatus which subjects an annular specimen to pure shear without imposing compound stress. The investigation was commenced by Professor C. F. Jenkin,[2] F.R.S., who after completing his work on the stability of dry, granular materials, turned his attention to the mechanics of clay materials. Unfortunately after certain work of a preliminary nature his health failed and he had to relinquish all connection with the research. Acknowledgement is here made to the valuable assistance which his technique and methods of approach to the subject have rendered to the present research.

Torsional Shear Tests.

The general layout of the apparatus is shown in *Fig. 1* (facing p. 336). The annulus of clay A is held between rings B and C whose faces are serrated with very small teeth. Ring C is fixed to a baseboard while B is free to rotate in a horizontal plane and can move vertically, its only restraint being a ball-race mounted at D which prevents translation in a horizontal plane.

A light pulley E is connected to the top ring B and transmits the torque to the ring. The pulley is connected to a cross bar G by light steel wires passing over ball-bearing pulleys F and F′. A plywood arm H resting on the pulley has a pointer at its extremity which records on the smoked plate J. A light tube K mounted at its lower end on a knife edge carries the smoked plate and is fixed rigidly to steel rods L and L′. The cross bar G is connected by a steel wire to hooks on the tube at M and the torque applied by means of the ball-bearing pulley N over which the steel wire passes.

When the apparatus is set up, the steel wires are just slack and the rotation of pulley N about a point in its plane causes the rods L and L′ to be stressed, this stress being transmitted to the beam G and thence to the specimen. The rods L and L′ are previously calibrated and both the applied torque and angular rotation of ring B are recorded on the smoked plate. The annular specimen

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1 A. L. Higgins, "The Wedge Theory in its Application to Clays and Cohesive Earths," Sessional Notices Inst. C.E., Dec. 1930, p. 57.

2 C. F. Jenkin, "The Pressure on Retaining Walls," Minutes of Proceedings Inst. C.E., vol. 234 (1932), p. 103.

has an outside diameter of 4 inches, an inside diameter of $3\frac{1}{4}$ inches, and a depth of from $\frac{3}{4}$ inch to $1\frac{1}{2}$ inch.

By counterbalancing the weight of the pulley E and pointer H torsion tests can be performed under zero normal load and there are no restraints at the inner or outer circumference of the specimen. The apparatus also has the advantage that the specimen is under visual observation throughout the test.

To calculate the shearing stress from measurements of the torque, two assumptions may be made :—

either (1) that the shear stress is uniform from inner to outer radii;

or (2) that the shear stress is proportional to the radius.

With assumption (1), taking a uniform stress q_1 :—

$$\text{Torque} = 2\pi q_1\left(\frac{b^3 - a^3}{3}\right),$$ where b denotes the outer radius, and a ,, ,, inner ,,

With assumption (2), taking "triangular" variation :—

$$\text{Torque} = 2\pi q_2\left(\frac{b^4 - a^4}{4b}\right).$$

Thus the relation between the values of the shear stress calculated on these assumptions is :—

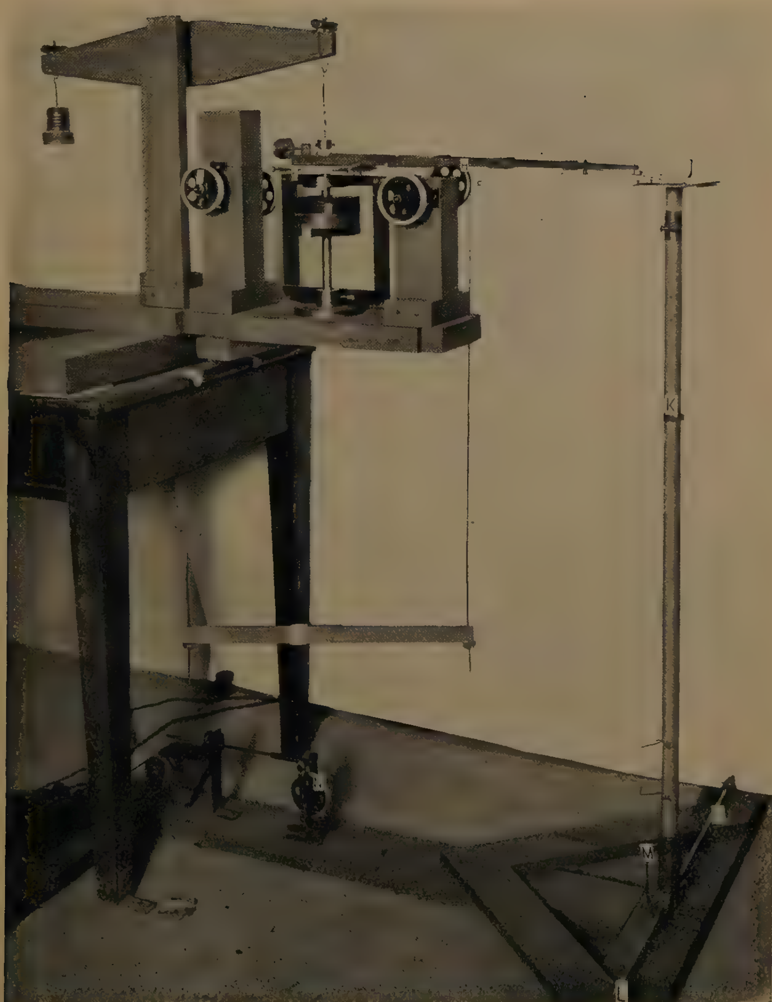
$$3q_2(b^4 - a^4) = 4q_1 b(b^3 - a^3).$$

In the present apparatus $b = 2$ inches and $a = 1{\cdot}625$ inch and hence $q_1 = 0{\cdot}912q_2$. It seems likely that the first assumption is sufficiently good, and the results quoted have been calculated on this basis. It may be pointed out, however, that whichever assumption is adopted for the calculation of the shear stress for a series of tests, only the absolute value of the shear stress is affected and the shape of the shear-stress/water-content curve is unaltered.

Experiments have been performed mainly on samples either remoulded from their natural state or prepared from the air-dried condition by thorough manipulation with different proportions of water. Undisturbed samples have also been tested by a technique which allows annular rings to be prepared and transferred to the apparatus without distortion of their form.

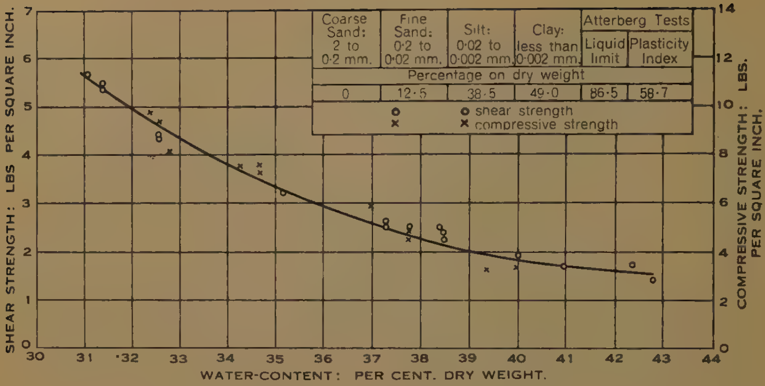
A number of typical clays have been tested in pure shear by this apparatus, including very "plastic" clays and those of a more sandy nature. The results have indicated the pronounced influence of the water-content on the ultimate shearing resistance, and isolated tests on samples of undisturbed soil have also shown the influence of the structure of the soil on its mechanical behaviour. It is hoped to investigate the influence of structure much more fully in the near future.

Fig. 1.



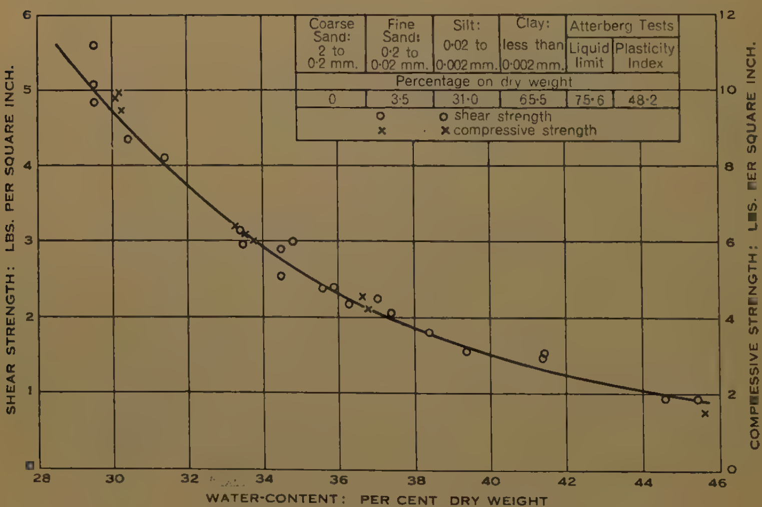
TORSIONAL SHEAR APPARATUS.

Fig. 2.



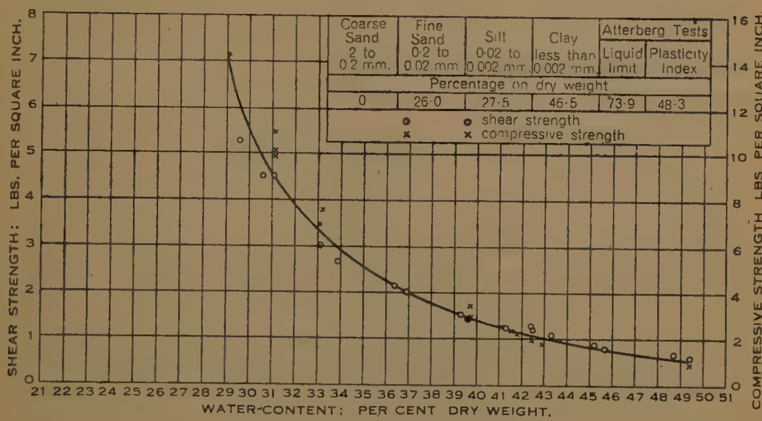
LONDON CLAY (SAMPLE No. 129.)

Fig. 3.



READING CLAY (SAMPLE No. 114.)

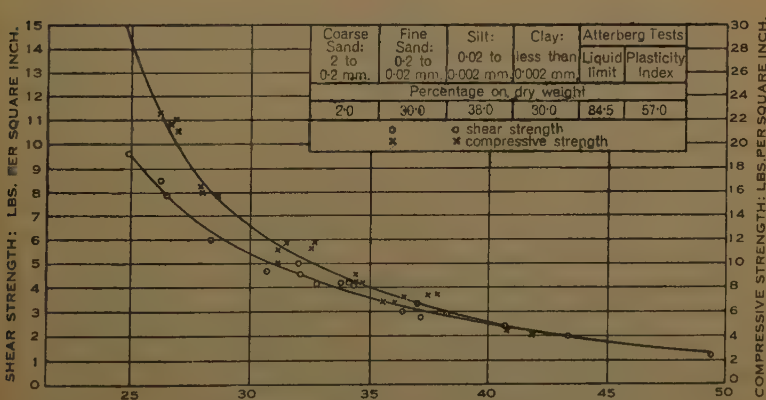
Fig. 4.



Gault Clay (Sample No. 108).

Figs. 2, 3, 4, and 5 show the results obtained for the variation of ultimate shear resistance with water-content for a number of typical samples of clays. Values for certain simplified physical tests are also included to give an idea of the character of each particular soil. The behaviours of different types of clay show interesting differences. With "plastic" clays the usual failure is along a horizontal plane through the middle of the annulus. In these cases the torque/strain curve presents a smooth, well-rounded appearance and the drop in torque after the maximum has been reached is quite gradual.

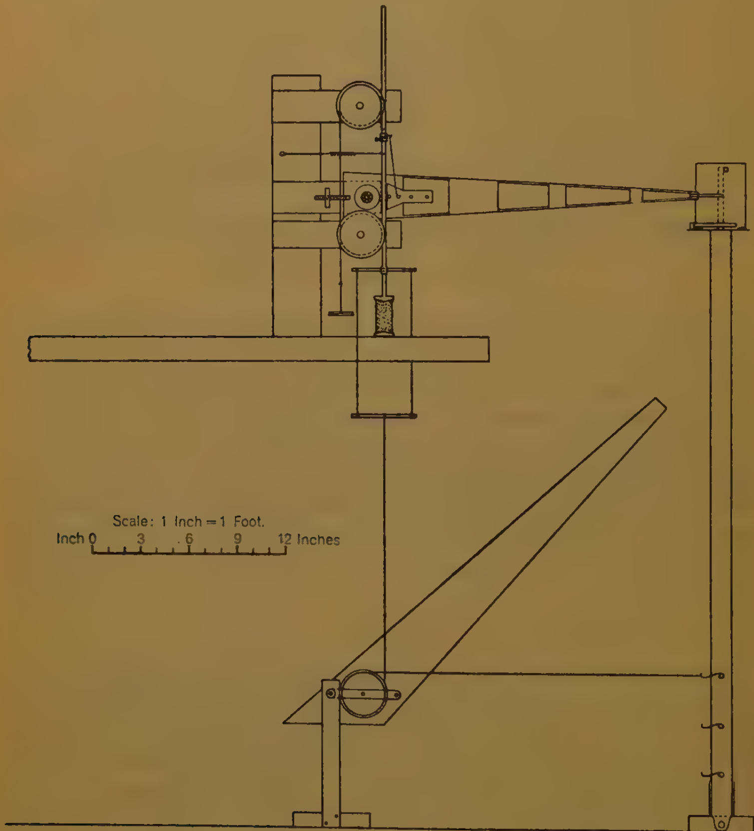
Fig. 5.



Bagshot Beds—Silty Clay (Sample No. 1348).

Sandy clays by contrast exhibit a failure by a shear plane which is usually inclined to the plane of rotation and there is a rapid drop in torque after failure has occurred. In both cases the failure plane extends right through the specimen from the inside to the outside circumference.

Fig. 6.



APPARATUS FOR COMPRESSION TESTS.

Normally the torque is increased at a uniform rate such that the time of test from start to failure amounts to approximately 3 minutes. Tests have been done at different uniform rates over a range such that the time of test varied from ½ minute to 70 minutes. The shape of the torque/strain curve is not the same for different speeds, since with a fast loading there is not sufficient time for all

the "plastic" deformation to occur. The value of the maximum torque, however, was not found to vary appreciably over the range of speeds examined.

Compression Tests.

In a recent paper L. Jürgenson [1] has pointed out that one of the most simple and reliable methods of investigating shear strength is by the axial compression of a cylinder laterally unrestrained. The failure produced in this test is not one of pure shear on account of the accompanying normal stress on the plane of shear, but nevertheless the relation between the results of shear strength calculated from compression tests and those obtained by the torsional shear method are of especial interest.

Compression tests have been performed both by a spring-loaded apparatus in which the load is increased by small increments corresponding with small increments of strain and by an apparatus illustrated in *Fig. 6*. Here the load is applied by a device similar to that used in the torsional shear apparatus and load/deformation curves are recorded automatically on a smoked plate.

Tests have been made using different-sized specimens and with different rates of loading, but for most tests a specimen of $\frac{3}{4}$ inch diameter and $1\frac{3}{4}$ inch long is used, the duration of a test varying between 3 and 5 minutes. In preliminary tests flat end-pieces were used, but "barrelling" of the specimens resulted and, in consequence, coned end-pieces were adopted. By using such coned end-pieces with suitable base angles of cone, varying between 5 and 15 degrees, the specimen remains approximately cylindrical throughout the test.

The form of failure which occurs varies with the type of soil and, to a lesser degree, with the moisture-content. In most cases a well-defined shear plane passes right across the specimen clear of the end-pieces, and in some cases shear planes occur inclined equally to and on opposite sides of the axis of compression. Sometimes Lüders lines are visible in early stages of the test and multiply rapidly. With "plastic" clays the deformation to failure is large and the compressive stress falls away slowly after the maximum has been reached. With sandy clays the strain to failure is small and rupture is of a brittle nature, the stress dropping rapidly after the maximum has been reached. This behaviour follows that exhibited by different clays under torsion.

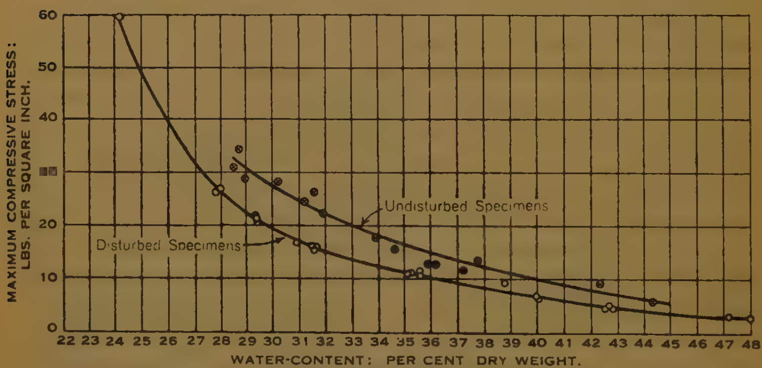
Observation of the angle at which the shear plane is inclined to

[1] L. Jürgenson, "The Shearing Resistance of Soils," Journ. Boston Soc. C.E., vol. 21 (1934), p. 242.

the axis of compression shows that it is 45 degrees or slightly less. In some cases angles measured in repeated tests have shown close agreement, but in the majority of cases they have not been sufficiently consistent to form a basis from which deductions could be drawn.

Experiments have been made on both disturbed and undisturbed samples. The remoulded samples were prepared by a similar process to that used for shear specimens and the results of tests on a number of typical clays are given in *Figs. 2, 3* (facing p. 337), *4* and *5* (p. 338), in which the compressive strength is plotted on twice the scale of shear strength. This comparison shows that with "plastic" clays the shear strength is equal to one-half the compressive strength. Since these clays are believed to possess negligible

Fig. 7.



RELATION BETWEEN COMPRESSIVE STRENGTH AND WATER-CONTENT FOR UNDISTURBED AND REMOULDED SAMPLES OF LONDON CLAY.

friction, this relation is of especial interest. With sandy clays the compressive strength is greater than twice the shear strength and this excess is more evident in the drier condition. The difference is probably associated with the internal friction, which in sandy clays may be appreciable, and a tentative theory for estimating the angle of internal friction from the relation between the compressive strength and the shear strength is being investigated.

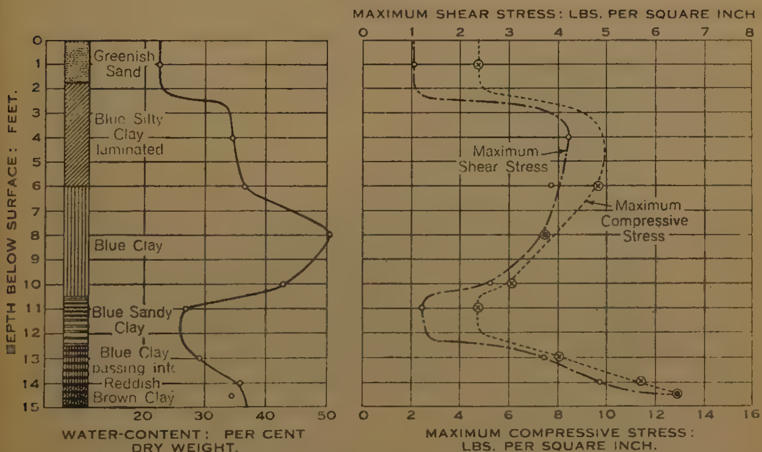
Tests on undisturbed samples have shown that the structure of the soil may have an important influence on its compressive strength. *Fig. 7* shows the variation of compressive strengths of a number of undisturbed samples of London clay with the natural water-content at which they were obtained. It also shows a similar relation for the clay in a remoulded condition.

Practical Application.

The methods of investigating shear strength described above have been applied in connection with the examination of the site of a recent landslip. Unfortunately, it was not possible to obtain undisturbed samples of the various soil-types occurring in the profile, but specimens for test were obtained by remoulding at the natural moisture-content.

The results obtained for the compressive strength (p) and the shear strength (q) of the various soil-strata are shown in *Figs. 8.* The modified Coulomb's relation was applied to the calculations of the stability of the slope and values of c and ϕ were deduced from

Figs. 8.



Compressive Strength and Shear Strength of Different Soil-Types in a Soil-Profile.

measurements of p and q. By assuming that the resistance to shear obeys the modified Coulomb's relation $t = c + n \tan \phi$,

then in the compression test $p = 2c \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$,

and in the shear test $q = c \cos p$.

Hence $p = 2q \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \sec \phi$ is the relation between maximum compression stress and maximum shear stress. Thus from measurements of p and q values of c and ϕ can be evaluated. It will be noted that when $\phi = 0$, $p = 2q$ and that as ϕ increases, p becomes greater than $2q$.

The following values were deduced for the four main types of soil encountered down the profile.

Soil type.	Cohesion: lbs. per square foot.	ϕ: degrees.
A	188	33
B	360	6
C	166	30
D	576	4

It may be mentioned that type C was of a sandy nature and in a very wet condition so that when remoulded both the cohesion and

Fig. 9.

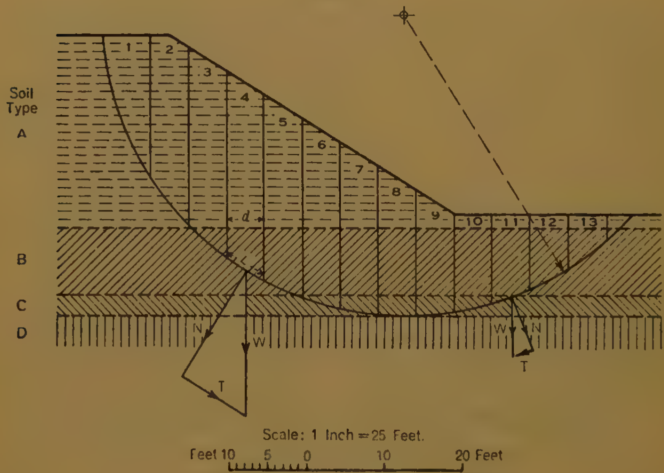


Diagram of Slip-Surface.

angle of internal friction probably became greater than they were originally in the natural state.

The method of analysing the stability of the slope was based on the cylindrical slip-surface theory originated by the Swedish investigator Petterson and later developed by Fellenius and Krey. An account of this and other methods will be found in the paper by Terzaghi.[1] The arc of a circle shown in *Fig. 9* represents the slip surface. Briefly the principle consists of considering the stability of the "cylindrical wedge" of earth against rotation about the axis of the cylinder. The forces causing rotation are thus due to

[1] K. Terzaghi, *loc. cit.*

the weight of the mass, and are resisted by the friction on the cylindrical surface and the cohesion of the soil.

This particular slope had been stable for years and the explanation was sought as to why it had suddenly become unstable as the result of recent rains. The analysis offered such an explanation. Under the moisture-content conditions of the various soil-types as measured from the samples taken, the disturbing turning moment due to the weight of the wedge was about $392 \times 10^2 \times R$ lb.-feet, where R is the radius of the cylinder of slip. The resisting moment under the same conditions was $661 \times 10^2 \times R$ lb.-feet, and thus stability should result. But it was also apparent that soil-type C was one whose properties were specially liable to be affected by the change in moisture-content, and in fact contributed $322 \times 10^2 \times R$ lb-feet as a friction resistance in the total resisting moment. If this stratum were flooded, its frictional resistance would be very much reduced indeed. Observations on the site during subsequent operations indicated the probability that this was the cause of failure.

REPORT OF THE NATIONAL PHYSICAL LABORATORY FOR THE YEAR 1935.[1]

A description of the activities of the National Physical Laboratory was given in the February issue of the Journal. In addition to their work in connection with the standardization and testing of materials, a wide range of subjects is covered by their researches and their report for the year 1935 deals with progress in such varied branches of physics and engineering as heat and refrigeration, sound, optics, radiology, electricity and magnetism, radio, fine measurements, mechanical engineering, metallurgy, aerodynamics and ship design. It is proposed to mention briefly some of the researches of greatest interest to engineers.

In the Physics department a study of the internal structure of metals by X-ray methods has been continued, and it has been possible to study the changes which occur in the metal beyond the elastic limit. Below this no change occurs, but above the yield point a broadening of the spectral lines indicates a breaking down of the crystals into a mosaic of component grains differing little in orientation. Only near the breaking stress does distortion of the lattice within the grains take place. A study has also been made of electroplated metals and surface structure. It is found that natural lustre is associated with a high degree of parallelism in crystals.

[1] Published by H.M. Stationery Office. Price 12*s*. net.

In continuing the investigation of sound transmission through buildings it has been found that there is little difference in the sound-transmitting properties of lime and of cement in mortar and plaster walls. Impact sounds are largely transmitted through floors, and the effect of insulating layers such as the underfelt below carpets and the construction of a floating floor supported by rubber pads has been investigated.

In the Electricity department researches on the behaviour of dielectrics at very high frequencies have been carried out. Use is being made of the cathode-ray oscillograph in high-voltage research, and the breakdown characteristics of dielectrics under surge-voltage conditions and the protection of apparatus from transient electrical stresses have been studied. In another research the efficiency of street lighting in regard to the illumination of the carriage-way is being examined.

In the Engineering department research on journal-bearings has been continued. A study of the minute changes of shape of a bearing during "running in" has been made. As a result of this research it has been found that very slight modifications of shape have a marked influence, and also that "running in" is more effective if the journal is rotated in both directions. In connection with road research a study has been made on an actual road of the effect on wheel impact of different tire systems, inflation pressures, shock absorbers and the like. Research on pipe flanges arranged in conjunction with the Institution of Mechanical Engineers has been continued and the tightness of joints under varying conditions of pressure and temperature has been studied. The first report of the Pipe Flanges Research Committee was presented to the Institution of Mechanical Engineers in February 1936.

The Metallurgy department, as the result of a study of the thin oxide films on metals by means of electron diffraction, has been able to obtain some insight into the liability of the metal to corrosion. The film on aluminium, which is most effectual in checking further oxidation, has been found to be really amorphous: most metals form crystalline oxide films. The quantity and distribution of the combined oxygen in steel is known to have a profound influence on the properties of the metal, and a method is being developed by which steel can be produced free from dissolved oxygen. A study of the behaviour of metals at high temperatures has been carried out in a vacuum in order to avoid complications due to the presence of oxygen.

In the Aerodynamics department a research has been made into the design of flaps at the rear edge of aeroplane wings to facilitate safe landing. The effects of gusts on aeroplanes is being studied,

and in this connection an instrument is being devised to measure the vertical velocities occurring in gusty weather. Special problems have arisen with the development of modern high-speed aircraft. An airscrew of high pitch suitable for high speeds is very inefficient when the machine is taking-off, and possible remedies such as varying the pitch and varying the speed of the airscrew are being investigated. The mechanism of flutter of aeroplane wings at low speeds is fairly well understood, but at high speeds the problem is more complicated and is being investigated.

WORK OF THE INSTITUTION OF AUTOMOBILE ENGINEERS RESEARCH LABORATORY.

A milestone in automobile research was passed on the 18th March, 1936, when Lord Rutherford, O.M., M.A., D.Sc., LL.D., F.R.S., declared open the new Research Laboratory of the Research Committee of the Institution of Automobile Engineers. The research had its origin in 1920, when the Research Association of the British Motor and Allied Manufacturers was formed as a Co-operative Industrial Research under the Department of Scientific and Industrial Research. In 1931 it was taken over by the Institution of Automobile Engineers under their Research and Standardisation Committee.

The equipment includes a chemical laboratory and a materials-testing laboratory, containing a 5-ton Buckton testing machine, an Avery fatigue machine, a Vickers projection microscope, a Vickers pyramid-hardness machine and sundry other apparatus. There is a chassis laboratory, equipped with dynamometers by means of which tests on complete vehicles can be carried out. In addition there is an engine laboratory containing eight test beds, a general laboratory, workshop and stores.

Many researches are in progress and their importance will be apparent to every motor user. A study is being made of the various factors which contribute to the wear of cylinders and pistons; and the effects of the previous mileage of the lubricating oil and of contamination by foreign matter are being examined. Cylinder wear is found to be accentuated under cold-running conditions such as obtain when starting up, and a study is being made of this. Experiments are being carried out on the effect on the wear during running-in of a new engine of the initial cylinder-bore finish (e.g. ground, fine-bored, mirror-finished, etc.). Cylinder wear at high temperatures is being studied in an air-cooled machine. Other factors such as upper-cylinder lubrication and width of piston rings are also being

studied. Valve-seat wear is being studied both under artificially-controlled conditions and in an actual air-cooled engine. The ability to start up an engine at low temperatures is closely related to the engine friction and, in order to study the power required to start up, tests are being carried out in a refrigerator at a temperature of $-10°$ C.

A study is being made of the effect of temperature and oil viscosity on the wear of the white metal linings to big-end bearings, and another research concerns fatigue cracking of white metal and other modern bearing metals under conditions analogous to those obtaining in big-end bearings. The wear, pitting and noise of gears composed of various steels and machined by various methods and the effect thereon of different lubricants is being studied in a special machine. Two pairs of gears are run in the form of a closed train, any desired load being introduced by uncoupling a flange, introducing an internal twisting moment and bolting up again. A study is being made of the corrosion of ball-bearings under various conditions of storage.

An investigation of brake-squeak is being carried out by driving a back axle at various speeds by means of a reduction gear with the brake full on. The vibrations at various parts can be studied by means of a cathode-ray oscillograph. Another research relates to the wearing and frictional properties of various brake-drum and brake-lining materials.

The following researches are being carried out for the Committee elsewhere where there are greater facilities for the particular researches. A study of the deep-drawing of sheet steel and methods of assessing the suitability of this process for various steels is being carried out at Sheffield and Swansea Universities. The fatigue strength under combined stresses and the suitability of cast materials for crank-shafts are being studied at the National Physical Laboratory. An investigation of lubricating-oil consumption, firstly with petrol and secondly with oil engines, is being carried out at the works of Messrs. Ricardo & Co., Ltd., and the impact of tyres on roads is being studied in conjunction with the Road Research Board at the National Physical Laboratory.

Further researches which are contemplated under the auspices of the Research Committee are a study of engine friction at high speeds and an investigation of mechanical noises in motor-cycles.

RESEARCH WORK IN ENGINEERING AT BIRMINGHAM UNIVERSITY.

(May 1936.)

A wide field of engineering is included in the purview of the Faculty of Science of the University at Birmingham, and a brief description of the researches in the departments of Civil, Mechanical and Electrical Engineering, Oil Engineering and Refining, Metallurgy, and Mining will be given.

Civil Engineering.

The laboratories of the Department are well adapted for research on structures and constructional engineering. The testing machines include 10- to 50- and 100-ton machines of various types, a 500-ton compression machine and a 300-ton machine capable of taking specimens of large size in tension, compression or bending. It is possible to test whole structures in this machine, and an example of such a test, on a full-scale steel frame, is referred to in the discussion on the Paper on the design of steel buildings in the present issue of the Journal. A large concrete-laboratory has also recently been built in which tests on full-sized reinforced-concrete structures can be carried out.

The most important researches undertaken in recent years are the investigations for the Steel Structures Research Committee, the results [1] of which are published in the three Reports of the Committee. These include researches on bolted and riveted joints and on the behaviour of beam-and-stanchion connections, and the evolution of a new method of design for beams. This work was carried out intensively for about 6 years, and it is proposed to continue the investigation on the full-scale steel frame, in order to study the effect of lateral loads. Research on riveted splices is also contemplated. A number of mathematical investigations on rigid frames [2] and flat plates are being carried out.

Experiments on composite members, consisting of steel beams encased in concrete, have been in progress for some years. These have been directed towards the determination of the fundamental principles underlying the action of such structures, and have included strain measurements, experiments on the conditions affecting bond, and several series of tests to determine the redistribution of stress

[1] See also p. 127 of this issue.

[2] E. H. Bateman, "The Stability of Tall Building Frames," Inst. C.E. Selected Engineering Paper No. 167. "The Open-Frame Girder," Journal Inst. C.E., vol. 1 (1935–36), p. 67. (November, 1935.)

and change of deflection with time. Experimental work is also in progress on the action of reinforced concrete in tension. A research on the properties of concrete [1] has recently been completed showing the effect of the characteristics of coarse aggregate on the properties of concrete mixes, of the time of mixing, and the influence of the rate of loading on the compressive strength.

Mechanical Engineering.

Many researches are being conducted on the internal-combustion engine. A research is being carried out on the combustion in high-speed solid-injection oil engines, and the effect of modifications to the inlet valve has been studied. The design of a satisfactory engine indicator for high-speed engines is rendered difficult on account of inertia effects. In an attempt to overcome these, gauges based on piezo-electric, electrical-resistance and other properties are being developed. Much has been achieved in the way of silencing of exhaust noise by the scientific design of silencers. At Birmingham the problem is, however, being tackled at its source and an attempt is being made to minimize the exhaust noise of a motor-cycle by suitable valve-timing.

A study of the interchange of heat when air at one temperature flows between parallel metal plates at different temperatures has been carried out. A further research concerns the cooling effect of an air stream impinging obliquely on a metal surface.

A study has been made of the embrittlement of steels due to soaking with water at high temperatures. An investigation into the drawing of sheet metals is also being carried out.

Electrical Engineering.

Michael Faraday carried out experiments with an axially-spinning magnet which appeared to indicate that the magnetic field remained stationary and independent of the magnet. This investigation into the axial spin of a magnetic field [2] is being extended with the immediate practical objective of developing a new and inexpensive permeameter. This is required by manufacturers of steel for magnetic purposes, as the present methods of determination of permeability are costly on account of the large samples required for test.

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[1] H. W. Coultas, "The Effect of the Coarse Aggregate and Other Factors on the Properties of Concrete," Journal Inst. Struct. Eng., vol. xiv (1936), p. 131.

[2] Professor W. Cramp and Dr. E. H. Norgrove, "Some Investigations on the Axial Spin of a Magnet and on the Laws of Electro-Magnetic Induction," Journal Inst. E.E., vol. 78 (1935), p. 481.

A research is being carried out on magnetization under combinations of direct and alternating current. The incremental magnetization and associated losses at all frequencies [1] are being studied with a view to the possibility of specifying the conditions under which measurement should be made. An associated research is the development of a method of harmonic analysis by synchronously-running rectifiers.

In connection with the Illuminating Engineering Society a research on the surface brightness of diffusing glassware is being undertaken in order that brightness may be predicted from a knowledge of the composition of the glass and the shape of the globe.

A study of the wear of trolley wires, in conjunction with the Electrical Research Association, is in progress. Tests indicate that copper is not the most suitable material for contact shoes.

At very low loads errors occur in induction watt-hour meters, and a study of these is being made.[2]

Oil Engineering and Refining.

The drilling strings used for deep bores may be regarded as very long columns with intermittent lateral support from the well-casing. They may also be regarded as long rotating shafts in which the critical speed occurs at an exceedingly slow rate of rotation. A study of these factors is being made with a view to ascertaining the conditions necessary for a bore to run straight. With boreholes in limestone formations a treatment by forcing down acid at a high temperature and pressure is found to increase the yield of oil from the hole.

Researches into the preparation of synthetic lubricating oils are being carried out from two angles, by means of condensation reactions and by synthesis of olefinic hydrocarbons in the presence of a catalyst.

A study is being made of the application of phase-rule principles to the de-waxing of lubricating oils and the sweating of paraffin wax.

Metallurgy.

A research is being carried out into the constitution and mechanical properties of alloys of tin with cadmium and with antimony. The addition of cadmium produces an alloy amenable to heat treatment in which the strength is increased and the creep reduced. Tin and

[1] L. G. A. Sims, "The Specification of Magnetic Qualities," *Engineering*, vol. cxl (1935), p. 290.

[2] T. Havekin, "A Study of the Induction Watt-Hour Meter, with Special Reference to the Cause of Errors on Very Low Loads," Journal Inst. E.E., vol. 77 (1935), p. 355.

its alloys are plastic at very low stresses, and long-term tests extending over from 1 to 2 years are being carried out to study the relation between creep and loading. Improved alloys have been discovered and are being fully investigated.

Research is being conducted on the properties of some complex bronzes, and on the distribution of unsoundness in aluminium alloys. The structure of metals and alloys is being analysed by a study of the electrical resistivity, and the crystal structure is being determined by means of X-rays.

Mining.

Of recent years attention has been directed to the more efficient utilization of coal,[1] especially the smaller particles from $\frac{3}{8}$ inch down to dust. In this connection there is being carried out at Birmingham an investigation into the fundamental principles of gravity-separation of minerals. As a result of research so far completed, two important structural additions to the washer-box mechanism have been designed and adopted for use in practise. An investigation is also being carried out on the clarification of washing water.

Another research concerns the design of mine-rescue apparatus and the problem of training the miner in its use. The energy output of miners [2] is found to depend on many factors. The effect on the output of working in different postures is being investigated. The efficiency of a man as a machine, that is, the relation between food consumed and the work done, is being determined.

The Mining Research Laboratory, till recently under the direction of the late Prof. J. S. Haldane, is attached to the Mining Department of the University. Research investigations are being conducted into the spontaneous combustion of coal; and the production and elimination of dust in mine atmospheres with a view to preventing silicosis.

The above researches are being carried out under the direction of Prof. C. Batho, D.Sc. (Civil Engineering), Prof. S. Lees, M.A. (Mechanical Engineering), Prof. W. Cramp, D.Sc. (Electrical Engineering), Prof. A. Nash, M.Sc. (Oil Engineering and Refining), Prof. D. Hanson, D.Sc. (Metallurgy), and Prof. K. Neville Moss, O.B.E., M.Sc. (Mining and Dean of the Faculty of Science).

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[1] Dr. F. S. Sinnatt, "Some Major Problems in the Utilization of Coal," Journal Inst. C.E., vol. 2 (1935–36), p. 545. (April, 1936.)

[2] Professor K. N. Moss, "The Energy-Output of the Coal-Miner," Journal Inst. C.E., vol. 1 (1935–36), p. 354. (January 1936.)

NOTES ON RESEARCH PUBLICATIONS.

Measurement, and Measuring- and Recording-Instruments.

The measurement of the distribution of particles of different sizes by an optical method is discussed in *Proc. Am. Soc. Testing Materials*, **35,** *457*. The calculation of loss of heat through furnace-walls is complicated by the variation of conductivity and temperature : a graphical solution is described in *Am. Ceram. Soc. J.*, **19,** *74*. The absolute determination of the ohm is described in *U.S. Nat. Bur. Standards J. Research*, **16,** *1*.

Engineering Materials.

In connection with researches into the properties of engineering materials the method of selecting and testing structural timbers, and British Columbian Douglas fir in particular, is dealt with in *Forest Products Research Records No. 8*. A study of the effects of coarse aggregate and other factors on the properties of concrete is given in *Structural Engineer*, **14,** *131*. Tests on the hardening of cement mortars during freezing are given in *Ton-Industrie Zeitung*, **59,** *1092*. Repeated freezing and thawing is found to increase the sizes of the pores of brick according to research published in *U.S. Nat. Bur. Standards Technical News Bulletin No. 220, 7*. A study of the factors influencing shrinkage in concrete is given in *Proc. Am. Soc. Testing Materials*, **35,** *383*. The properties of pure iron and the method of preparation are described in *U.S. Nat. Bur. Standards J. Research*, **16,** *105*. A report on the investigation of fissures in railway rails is given in *Univ. Illinois Eng. Expt. Stn. Bulletin*, **33,** *16*. In *Franklin Inst. J.*, **221,** *485*, random fracture of a brittle solid is discussed and the occurrence of cracks and fissures studied by statistical methods. A photo-elastic method of studying stresses due to impact is given in *Phil. Mag.*, **21,** *448*. In the same volume, *p. 65*, it is shown that at small stresses a transient value of the modulus of elasticity above the normal has been found in connection with compressive tests on steel, glass, stone and concrete. It is stated in *J. Roy. Tech. Coll. Glasgow*, **3,** *636*, that continuous abrasion and corrosion acting simultaneously have a mutually inhibiting effect due to the reduction of abrasion to polishing owing to the lubricating action of the corrosive fluid. In tests on continuous steel beams it is found, according to *International Assoc. for Bridge and Struct. Engg.*

The figure in heavy type is the number of the Volume ; the figure in brackets the number of the Part ; and that in italic type the number of the page.

Publications, **3**, *222*, that failure occurs in successive layers with a series of jerks as yield of the steel takes place.

The production, manufacture and preservation of materials are dealt with in the following references: non-pressure methods of applying wood preservatives are dealt with in *Forest Products Research Records No. 9*. A method of minimizing wood shrinkage by replacing water by an intermediate liquid such as cellosole, and this in turn by a wax or resin, is described in *Industrial & Engg. Chemistry*, **27**, *1480*. Results of research on the carbonation of unhydrated Portland cement are given in *Building Res. Stn. Tech. Paper No. 19*, and the seasoning of Portland cement at elevated temperatures is dealt with in *Industrial & Engg. Chemistry*, **28**, *362*. Reduction of strength has been found to occur in mortars heated to 60–70° C. according to *Zement*, **24**, *749*. The effect of curing temperature on the compressive strength of concrete at early ages is discussed in *Am. Concrete Inst. J.*, **7**, *212*. The corrosion of cement by water containing carbonic acid is dealt with in *Soc. Chem. Ind. J.*, **55**, *117*. A study of the relation between quality of concrete and economy is contained in *Franklin Inst. J.*, **221**, *495*, and the same subject is also dealt with in *Am. Concrete Inst. J.*, **7**, *459*. The report of the American Concrete Institute on the placing of concrete by vibration is given in *Am. Concrete Inst. J.*, **7**, *445*. Research on the crazing of cast stone and concrete carried out at the Building Research Station is described in *Concrete Building & Concrete Products*, **9**, *10*, and recent developments in the manufacture of cast stone such as the use of vibration and casting of the face in superior material are discussed in *Am. Concrete Inst. J.*, **7**, *473*. As a preventive measure against attack by marine borers such as the *Teredo*, arsenic cement has been developed as a coating to timber piles, etc. Particulars of this process are given in *Perm. Int. Assoc. Navigation Congresses*, **11**, *93*. Research on the durability of concrete pipes is described in *Betong*, *1936* (*1*), *20*. The caustic embrittlement of steel at elevated temperatures, as in steam boilers, is found to be accelerated by the presence of sodium silicate in the water according to *U.S. Bur. Mines Tech. Pub. 691*. Research on the protection against corrosion of electro-deposited zinc and cadmium coatings on steel is described in *U.S. Nat. Bur. Standards J. Research*, **16**, *185*. Corrosion of steel by anaerobic bacteria is described in *Comptes Rendus* (1935), **201**, *1045*. The technical publications of the *International Tin Research Development Council, Series A*, include No. 31, the constitution of the tin-rich antimony—tin alloys; No. 32, influence of surface cuprous oxide inclusions on the porosity of hot-tinned coatings on copper; No. 33, the hot-tinning of copper: the attack on the basis metal and its effects. The results of research into

metallic corrosion in connection with aeroplane construction are given in *Revue de Métallurgie*, **33**, *145*.

Structures.

In connection with research on mass structures, a method of predicting the settlement of fills placed on soft ground by the carrying out of compression tests on samples is stated, in *Public Roads*, **16**, *249*, to give good agreement with the actual results. A method for the predetermination of piling requirements for bridge foundations by driving a steel rod into the ground has been developed at the Ohio State University Engineering Experiment Station (*Bulletin No. 90*). Tests on the strength of spirally-reinforced piles carried out at the University of Brussels are described in *La Technique des Travaux*, **12**, *151*. The uplift pressure on weirs can be reduced by the driving of a line of sheet piling at the upstream edge, and the effect of the length of the piling as studied both experimentally and theoretically is described in *Proc. Indian Acad. Sciences*, **2**, *646*. *L'Ingegnere*, **14**, *121*, contains a discussion of various methods of calculating the pressure on the linings of tunnels. Researches on the penetration of moisture into masonry walls are described in *U.S. Nat. Bur. Standards Technical News Bulletin, No. 226, 9*, and the efficacy of various finishes and paints in preventing the penetration of rain into concrete masonry walls is discussed in *Am. Concrete Inst. J.*, **7**, *485*.

The following notes refer to framed structures: a new code for building regulations for reinforced concrete, as revised and tentatively adopted by the American Concrete Institute, is given in its *Journal*, **7**, *407*. A report on the progress of a research on wind-bracing in steel buildings is contained in *Proc. Am. Soc. C.E.*, **62**, *397*, and a study of the stresses in tall building frames is described in the same volume, *p. 3*. A new formula for the carrying capacity of concrete columns has been evolved from research carried out at Lehigh and Illinois Universities and is described in *Eng. News-Record*, **116**, *361*. An investigation has been carried out at the Royal Technical College, Glasgow, on the distribution of the load among the rivets in joints containing several rows of rivets, and is described in its *Journal*, **3**, *599*. It is found that successive slipping takes place as the load is transferred from row to row. Theoretical and graphical methods for the analysis of multiple-span rigid-frame bridges by the slope-deflection method are given in *Am. Concrete Inst. J.*, **7**, *495*. A theoretical investigation of the stability against buckling of a rectangular slab subjected to non-uniform edge loading is described in *International Assoc. for Bridge and Struct. Engg. Publications*, **3**, *1*; in the same issue, *p. 271*, consideration is given to the buckling

strength of compression members in through lattice-girder bridges, and on p. *160*. the deflection theory for suspension bridges with two-hinged stiffening girders is discussed. A mathematical investigation of the direct and bending stresses in hemispherical dished ends such as are welded on to high-pressure cylinders is described in *J. Roy. Tech. Coll. Glasgow*, **3**, *609*. A paper read before the Institution of Chemical Engineers on 22 April, 1936, by D. M. Newitt, deals with the design of vessels to withstand high internal pressures. The theory of thick-walled and combined-walled vessels is discussed and allowance made for the overstressing of parts of the vessels, and the design of experimental apparatus to withstand a pressure of 15,000 atmospheres (100 tons per square inch) is described. A paper on the notion of generalized plane stress as applied to a flat plate of isotropic material of uniform thickness stretched or bent by forces applied to its edge is contained in *Phil. Mag.*, **21**, *201*. *Air Ministry, Aeronautical Res. Committee Reports & Memoranda, No. 1667*, gives a distribution method of stress-analysis.

Transformation, Transmission and Distribution of Energy.

The following electrical researches have been noted: in *Revue Générale de l'Electricité*, **39**, *407*, a new application of the rectifying valve is described. A research into electrical-resistance alloys of copper, manganese, and aluminium is described in *U.S. Nat. Bur. Standards J. Research*, **16**, *149*. *Electrical Engineering*, **55**, *245*, contains a paper on sinusoidal travelling waves. The theory of transients arising from the sudden application of a sinusoidal potential is presented for the case of a distortionless transmission-line and has been corroborated by experiments.

Mechanical Processes, Appliances and Apparatus.

A mathematical analysis of the stress in a strip due to variation of temperature along the length and through the thickness, such as occurs when heating takes place in a narrow band, as in welding operations, is given in *Physics*, **7**, *156*. In *Bautechnik*, **14**, *3*, tests to failure of welded girders under repeated loading show that the failing load is of the same magnitude as if calculated on the assumption of full yield-stress in compression above and full yield-stress in tension below a neutral axis. A research into the theory of film lubrication is described in *Annales de l'Association des Ingénieurs Sortis des Écoles Speciales de Gand* **26**, *3*.

Specialized Engineering Practice.

Among researches relating to transport may be mentioned experimental work on highways described in *Roads and Road Constrn.*,

15, *76*, which deals with work carried out in connection with the Road Research Board on bituminous surfacings, concrete foundations and surfacings, and surface dressings and thin carpets. In the Report on the 15th Annual Meeting of the U.S. Highway Research Board, 1935, the use of high-elastic-limit steel as reinforcement for concrete is discussed. The results of model tests on the resistance of ships having bulbs at bow and stern are given in *Schiffbau*, **37,** *55*. In *U.S. Nat. Bur. Standards J. Research*, **15,** *591*, accelerated service tests of pintle bearings such as are required for lock gates, which are being carried out in fresh water, are described. Phosphor-bronze was found to be the only satisfactory cup material and lubricating grooves were found to be advantageous. Report No. 771 of the U.S. Geological Survey gives a comprehensive record of available information relative to floods in the U.S.A., and No. 772 gives a detailed analysis of information relative to the relation between rainfall and run-off. Measurements relating the heights of ocean waves to the stresses in a ship are described in *Ver. deut. Ing.*, **80,** *208*. In *Mechanical Engg.* (*U.S.A.*), **58,** *215*, is a study of the vibrations and failures of aeroplane propeller crankshafts. A research on the factors involved in the flight of a rocket which resulted in the attainment of a height of 7,500 feet is described in *Smithsonian Misc. Collections*, **95,** *3*.

Research on hydraulics and water-supply includes a fundamental research carried out at the Kaiser-Wilhelm Hydraulics Laboratory on the measurement of the resistance to flow of rough surfaces, in which roughening in the form of regular excrescences of definite shape is employed; this is described in *Werft-Reederei-Hafen*, **17,** *99*. A study of the flow over horizontal broad-crested weirs with a view to their use for measuring discharge is given in *Civil Engineering* (*U.S.A.*), **6,** *257*. *Le Génie Civil*, **108,** *403* describes experiments on the piers of overflow spillways as a result of which it is found that with piers of aero-dynamic profile the reduction of discharge is negligible. In *Ohio State University Engg. Experiment Station Bulletin No. 87*, an investigation of the strength of bell and spigot joints in cast-iron pipes is described, and in *Bulletin No. 89* are given the results of research into the gauging of steam and water by means of venturi orifices.

In connection with researches on lighting, heating, ventilation and acoustics, the advances made in regard to electric illumination during the period 1932–35 are reviewed in *Inst. Elec. Eng. J.*, **78,** *171*. In *Univ. Illinois Eng. Expt. Stn. Bulletin No. 33*, *32*, a study of the conditions of temperature and humidity in buildings which give the maximum comfort in summer and winter is made; and a method of determining the thermal conductivity of insulating

materials such as asbestos, sawdust and cotton-wool is described in *Proc. Physico-Mathematical Soc. of Japan*, **17**, *321*. The thermal conductance of a brick wall is dealt with in *Inst. Heat. & Vent. Eng. J.*, **3**, *487*, and the thermal properties of concrete construction in *Heating, Piping and Air Conditioning*, **8**, *53*. A paper by the late Professor J. S. Haldane, F.R.S., on the ventilation of tunnels is given in the *Inst. Heat. & Vent. Eng. J.*, **4**, *18*, in which special reference is made to the researches in connection with the Mersey tunnel. A review of the progress in the theory of sound-absorption through porous walls is described in *Elektrische Nachrichten-Technik*, **12**, *333*.

NOTE.